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SUSPENSION BRIDGES—A STUDY.

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WITH DISCUSSION.

Fifty years ago the suspension bridge was regarded as the one class of structure adapted to spans of unusual length. Highway suspension bridges were built in almost all parts of the world; several railroad suspension bridges were proposed, and the one actually built across the Niagara River has done service for nearly forty years. Fifty years ago metallic bridge construction was in its infancy, and anything beyond the limit which could well be built of wood was considered an exceptional span. Athough there were a few striking exceptions, 200 feet was practically the limit of wooden truss bridge spans.

The introduction of iron bridges changed these conditions, and a 400-ft. iron span was as readily built as a 200-ft. Howe truss. The

first 400-ft. span in America was constructed by Albert Fink, Past-President Am. Soc. C. E., in the bridge across the Ohio River at Louisville, Ky., where it is still in use. The cheapening of the price of iron, the increased capacity of rolling mills, and the new methods of making steel, have rendered it an easier task to build a truss of 600 ft. span now than it was to build a 400-ft. span then. The result of this development has been that trusses have superseded suspension bridges, and where a suspension bridge would have been built forty years ago a steel truss is built now.

Furthermore, the old suspension bridges were highway bridges, and highway traffic does not enter upon a bridge as a concentrated load, while it is generally light in proportion to the dead weight of the bridge. Important modern bridges are generally railroad bridges, and the suspension bridge has not been considered stiff enough to serve this purpose, this want of stiffness being largely due to the fact that on a single-track railroad bridge the whole moving load comes on as a concentrated load, and that on railroad bridges of moderate span the moving load is large in proportion to the dead load.

No important suspension bridge has been built since the East River Bridge, the design of which was shaped by the elder Roebling before his death in 1869. During this time great advances have been made in all other forms of bridge construction. While it is admitted that there is no field for suspension bridges of the dimensions of which they were formerly built, a new field is opening in structures of enormous size, bridges that would bear the same relation to a 600-ft. steel truss that the 1 000-ft. suspension span at Cincinnati bore to the 200-ft. truss spans built about the same time in the same neighborhood across the Great Miami.

If, at the present day, an 800-ft. span is to be built, a truss on the beam or cantilever principle would be used, but a 2 000 or 3 000-ft. span would be a case for a suspension bridge. Two things must be remembered. In a suspension bridge of any such enormous size the dead weight would be large in proportion to the live load, and the distortion due to the passage of trains would be comparatively small. Such a bridge would be built for two or more railroad tracks, its length would be much greater than that of a single train, and the conditions under which a concentrated maximum load would advance as a whole, upon the bridge would be rare. In the manner of the passage of loads

such a bridge would resemble a highway bridge more than a short-span railroad bridge.

Before, however, a suspension bridge is built which will bear the same relation to the modern steel truss that the old suspension bridge did to the wooden truss, the same advance in details must be made in suspension bridges that has been made in truss bridges. If such a bridge is to be built now, the designer must concentrate in the work of a single design all the improvements corresponding to those which truss bridge builders have spent many years in developing.

This paper is submitted with a view to opening the way for improvement and to show that a great suspension bridge, which would be well adapted to railroad service, would involve no insurmountable difficulties of construction.

The method of demonstration and illustration which has been adopted is the explanation of the design of a suspension bridge of unusual dimensions and capacity. The size selected for this design would give a clear opening of about 3 000 ft., this corresponding to the dimensions proposed for the North River at New York. The plan discussed is simply a general plan, but as such a discussion to be valuable must be accompanied by estimates, the depth to rock at the sites of the towers has been assumed to be 140 ft. below mean high water; this corresponds to the depths which borings have found opposite the foot of Seventieth Street. It has been assumed also that the anchorages would be built on rock, and elevations have been assumed for these anchorages. These elevations correspond with reasonable accuracy to the position of the bluffs on the west side of the river opposite Seventieth Street, and are probably not very different from the position of the rock on the east side at the same location, but it is not likely that a structure located there would be of the symmetrical character which is now described. The design gives a clear headroom of 150 ft. at all stages of water.

While these conditions correspond more closely to the location at the foot of Seventieth Street than to any other place, this paper is intended as a study of suspension bridges and not as an approval of any particular location. The location at Seventieth Street undoubtedly has great advantages, especially in the matter of anchorages. A location below Thirty-fourth Street has also very great advantages, particularly in the matter of convenience to existing business centers.

It must be observed that the distinctive feature of a suspension bridge lies in the fact that it has no compression member. The weight is carried by cables stretched from tower to tower, which are secured by heavy anchorages at each end. The strains in the cables tend to overturn the anchorages, and any motion of the anchorages disturbs the entire structure; a slight settlement in the towers might not do much harm; any motion in the anchorages is felt through the entire bridge. In this respect the conditions of a suspension bridge are the opposite of those of a cantilever bridge. Any settlement in the towers of a cantilever bridge is exaggerated at the ends of the cantilevers, while the anchorages are usually of such character that they can easily be adjusted. It is therefore of the utmost importance that the anchorages of a suspension bridge should have foundations which will not yield and which can be put in at reasonable cost. These conditions are assumed in the estimates in this paper. A study of the paper, however, will show that the form of anchorage proposed admits of great latitude in use, as the inclination of the backstays becomes independent of the inclination of the main cables, and the anchorage for one pair of cables may be placed much farther back from the tower than the other pair. This might present great advantages if it became necessary to locate anchorages in a portion of the city already built up.

The capacity of the bridge designed has been reached in a back-handed manner. The bridge has been designed to carry a total load of 25 tons or 50 000 lbs. per lineal foot. The design has then been developed and the dead weight calculated, and the result is a balance for the live load of 11 000 lbs. per foot over the entire structure. As the width of the structure is 92 ft. between the stiffening trusses, this corresponds to about 120 lbs. per square foot of floor. If this space were to be occupied by eight railroad tracks it would amount to 1 375 lbs. per lineal foot per track, which exceeds the weight of any passenger train. It would amount in the aggregate on a length of 3 100 ft. to 34 110 000 lbs., equivalent to eight freight trains 1 400 ft. long, each weighing 3 000 lbs. per lineal foot. It is probable that the requirements of any location where a bridge of this magnitude would be considered, would be satisfied by four railroad tracks adapted to a heavy class of traffic and four rapid transit tracks to be operated by electric cars or short trains of a character which would require only a floor stiffener to secure the necessary rigidity. Therefore, in propor-

tioning the stiffening truss the variable load has been taken on the basis of 12 000 lbs. per lineal foot, corresponding to a load 3 000 lbs. per foot on each of the railroad tracks, with no provision for unequal weight on the rapid transit tracks, or to 1 500 lbs. per lineal foot on all eight of the tracks. These provisions correspond to four maximum freight trains or eight maximum passenger trains.

In one respect the design departs radically from suspension bridges hitherto built. The cables, instead of being made of straight wires, are made of ropes, and these ropes, instead of being passed over the towers and around pins in the anchorages, are socketed, both at the top of the towers and in the anchorages, all connections being made through the sockets. This modification is really the essential feature of the whole design. The objections which will be raised to it are, first, that a straight wire is both stronger and less extensible than a twisted rope made of the same wire; and second, that no socket can be made which will develop the full strength of the rope. Both of these objections are true, but a rope can be laid in such a way that the modulus of elasticity is only about 1 000 000 lbs. less than that of a straight wire, and a rope can be socketed in a way which can be absolutely depended upon to a fixed amount of strain, and the strength of the structure will then be determined, not by the strength of the wire, but the strength of the connection at the end of each rope. Furthermore, experiments have shown that ropes constructed in the manner proposed have an extremely uniform modulus of elasticity, which is the most important thing. The advantages of this system of construction are principally two; the ropes can be made in the shop, adjusted to length there, carried to the bridge site and put up in the least possible time; the wires are practically straight from one end to the other, the decided turns required over saddles and the short turns required around pins being entirely avoided. With this arrangement the objections to a strong stiff wire are removed.

Another feature which is believed to be novel is the method of holding down the ends of the stiffening truss. When one-half the span is loaded, the upward reaction at the unloaded end is equal to the downward reaction at the loaded end, so that the stiffening truss must not only be supported but anchored down. The stiffening truss of this design is made 1 000 ft. longer than the span, thus extending 500 ft. back toward the shore from each tower, while the suspenders in the

150 ft. next to each tower are omitted. The result is that the duties of the stiffening truss proper are confined to a length of 2 800 ft.; back of each tower is a span of 500 ft., from which a cantilever 150 ft. long projects to each end of the stiffening truss proper. The reactions of the stiffening truss are taken by the ends of the cantilevers, and the cantilevers are themselves anchored by the weight of the shore spans. This arrangement has the further advantage of leaving 150 ft. between the towers and end suspenders, within which the cables will adapt themselves to any changes of length and height due to temperature, loads or otherwise.

The description of the design follows the order of computation, the cables being first proportioned to carry the selected weight (50 000 lbs. per foot), the towers being proportioned to carry the cables, the foundations being proportioned to carry the towers, the anchorages designed to resist the pull of the cables, and the suspended superstructure to distribute the moving load.

General Design.

The general design is that of a stiffened suspension bridge, the cables to be of wire, the towers of steel on masonry foundations, the structure being stiffened by steel trusses suspended from the cables.

The cables are four in number, two on each side, the length of span between the theoretical intersection points on the top of the towers being 3 200 ft. and the versed sine 400 ft. To secure lateral stability, the two stiffening trusses are placed 100 ft. between centers horizontally, this affording an opening 92 ft. wide in the clear. At the middle of the span the two cables on each side are brought as close together as possible, or 4 ft. between centers, the width at the center of the span between points midway between the two cable centers being 115 ft. At the top of the towers the cables are spread apart to a distance of 28 ft., the width between the centers of the towers being 200 ft. Each inside cable has, therefore, a cradling of 30.5 ft., and each outer cable a cradling of 54.5 ft., the average cradling between 42.5 ft. The backstays are carried from the towers to the anchorages in planes which are tangent to the horizontal projection of the cradled cables, thus splaying the backstays apart between towers and anchorage. By this arrangement the towers are relieved of all transverse strain, and become, as it were, simply gin poles to

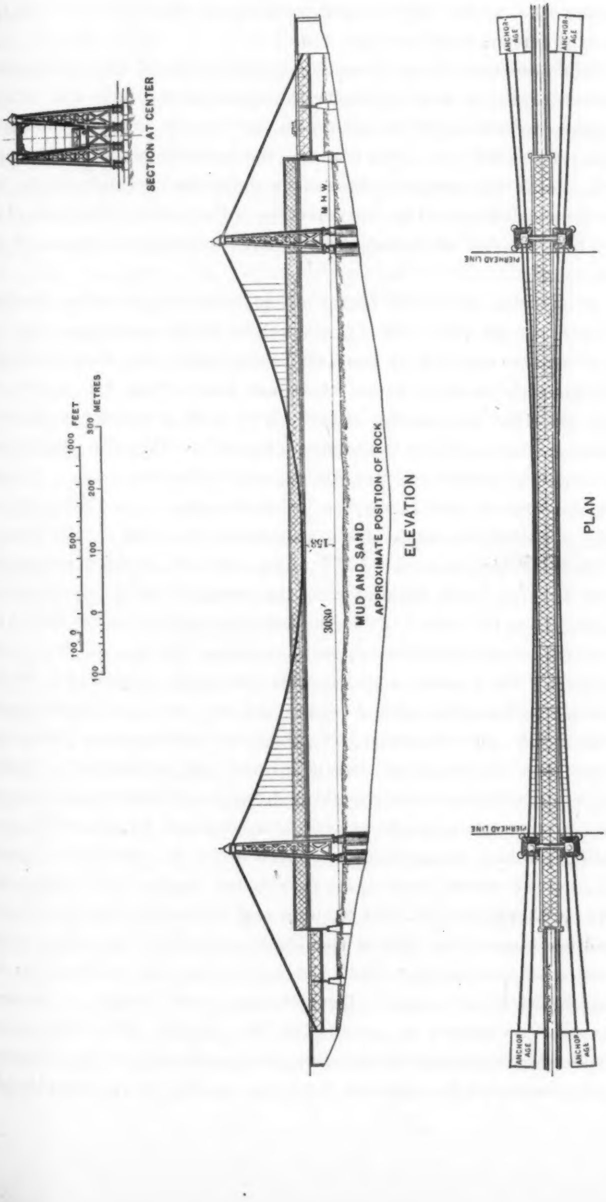

SECTION AT CENTER
FEET
METRES
100
0
500
1000
100
0
100
200
300
M H W
PIERHEAD LINE
PIERHEAD LINE
158'
3080'
MUD AND SAND
APPROXIMATE POSITION OF ROCK
ELEVATION
ANCHORAGE
ANCHORAGE
ANCHORAGE
ANCHORAGE
PLAN

Fig. 1.

sustain the cables. The lateral stability produced by this arrangement is evident from the plan, Fig. 1.

The towers are of steel, each really consisting of two independent towers formed of four posts, 94 ft. square at the base and battered together so as to be 28 ft. square on the top, the two square towers being connected by a cross truss at the top and resting on masonry cylinders at the bottom. The exact shape and location of the half towers are determined by the direction of the cables, the sides of the two half towers not being parallel and the squares being only approximate.

It has been considered important to reduce the number of cables to four, two on each side. If the cables of the main span and the backstays are counted as separate cables, which the detail hereafter described shows them to be, there are four cables, two leading in each direction, terminating at the top of each tower, the number of cables corresponding to the number of posts, so that the weight from each cable is transferred directly to one of the four posts. This requires cables of much larger dimensions than have ever been used, but there is nothing impracticable in making cables of the required size.

The stiffening truss is 4 100 ft. long over all, divided into panels of 33 ft. 4 ins. each, supported for the central 2 800 ft. by suspenders leading from the cables, while the ends are supported on piers 4 100 ft. apart and intermediate supports are taken on rocking bents 3 100 ft. apart. The truss is continuous for its whole length of 4 100 ft., fastened to the cables at the center and free to move longitudinally at each end. It is considered of great importance to use a continuous truss, thus avoiding the difficulties and lost motion of a central hinge. The difficulties of fastening down a stiffening truss are overcome by the end supports, the end of each truss being a 500-ft. span resting on two supports, from which a 150-ft. cantilever projects towards the point where the suspenders begin. The suspended stiffening truss is only 2 800 ft. long and exerts an upward or downward reaction at the end of the 150-ft. cantilever, according to the position of the moving load; the cantilevers are anchored by the weights of the end spans. The stiffening truss is 66 ft. 8 ins. deep between the centers of gravity of the chords, this depth being adopted for reasons given hereafter; it at once secures the necessary rigidity and permits sufficient flexibility to allow a considerable por-

tion of the irregular moving load to be taken care of by the change of shape in the cables.

The general elevation, Fig. 1, shows a clearance of 158 ft. above mean high water at the center of the span at a mean temperature of 60° Fahr., when the bridge is unloaded. The extreme effects of temperature are to lower or raise the center 3.15 ft., so that the maximum clearance of the unloaded bridge at a temperature of 0° Fahr. would be 161.15 ft., and the minimum clearance at a temperature of 120° Fahr. would be 154.85 ft. A moving load of 11 000 lbs. per lineal foot over the whole span, being the amount hereafter calculated on, and using a modulus of elasticity of 27 000 000 in the cables, will cause a deflection of 3.96 ft. at the center, thus reducing the minimum possible clearance to 150.89 ft. This load is, however, excessive, and the maximum load which should be estimated in calculating clearances is 9 000 lbs. per lineal foot, which will cause a deflection of 3.24 ft., making the minimum clearance 151.61 ft. The bridge is designed with a camber of 6 ft. in the center of the span when unloaded at mean temperature; this corresponds to a camber of 9.15 ft. when unloaded at the lowest temperature and to a camber of −0.39 ft. when fully loaded at the highest temberature. The maximum camber curve corresponds to a grade of 1.18% at the ends; but this is so short that it will not affect the passage of trains.

In this design the essential difference between a great suspension bridge and a truss bridge of ordinary dimensions has been borne in mind. The truss bridge of ordinary dimensions is so nearly a rigid structure that the changes which take place in its form under passing loads have little influence on the strains in the several members, and such a bridge can be proportioned on the basis of a rigid geometrical skeleton. A long span suspension bridge necessarily changes its shape with every change of load, and changes too in such manner as to relieve local strains, every unstiffened suspension bridge having some shape of perfect equilibrium for every possible loading. These changes of shape play an important part in proportioning a suspension bridge, and so long as they are kept within limits which do not disturb convenience of operation, they are a source of strength instead of weakness. A suspension bridge must be permitted to change its shape within proper elastic limits, and this change of shape must be made the basis of calculations in proportioning the structure.

Capacity.

The bridge has been proportioned to carry a total load of 50 000 lbs. per lineal foot, which is equivalent to a stress of 40 000 000 lbs. on each of the four cables at the center of the span. The actual dead weight of the cables and suspended superstructure is about 39 000 lbs. per lineal foot, thus leaving 11 000 lbs. for moving load.

The width in the clear between trusses is 92 ft., which will provide for two double-track railroads, each occupying 26 ft., with a space 40 ft. wide between. This 40 ft. can be occupied in various ways; its width is the same as the width between the curbs of Broadway at Twenty-sixth Street; it could be used for four rapid-transit tracks either for street cars or for rolling stock of the same dimensions as that used on the elevated railroads; it could be used as a street with two sidewalks and a roadway between wide enough for four carriages to pass; it could be used for two standard gauge railroad tracks, with a broad promenade for foot passengers between, or it could be used as a driveway with a street railroad track on each side. Another possible arrangement is the construction of eight parallel railroad tracks 11 ft. between centers which is admissible on perfectly straight lines. It is not important to decide how this bridge would be used; enough has been said to show the capacity which would be afforded, and the weights for which it should be designed.

Cables.

The cables are the fundamental feature of the design, and will therefore be described first. The design of the cables departs radically from the features hitherto followed in suspension bridges, and provides a method of constructing suspension bridge cables, under which it is possible to do nearly all the work in the shops, and to diminish field work to a minimum.

The bridge is designed with a versed sine of 400 ft. under maximum load, this being equal to one-eighth the span. On the basis of a uniform load of 50 000 lbs. per lineal foot, or 12 500 lbs. per cable, the stress in each cable becomes 40 000 000 lbs. at the center of the span, and 40 000 000 lbs. multiplied by $\frac{\sqrt{5}}{2}$, or 44 721 360 lbs. at the ends of the span, while the vertical reaction at each end is 20 000 000 lbs.

Each cable is composed of 253 ropes of equal size arranged in the form of a hexagon with three ropes omitted from each corner; the maximum stress on each rope will therefore be 176 764 lbs. In the design, the ropes are made $2\frac{1}{8}$ ins. diameter, each rope being assumed to have a section equivalent to 3 sq. ins. of solid metal and to weigh 10 lbs. per lineal foot. The stress per square inch on these ropes, will, therefore, be 58 921 lbs., of which $\frac{39}{50}$, or 45 958 lbs., will be caused by dead load, and $\frac{11}{50}$, or 12 963 lbs., by moving load.

Each rope will be a specially laid rope formed of a single straight wire at the center, around which are grouped successive layers of helicoidal wires, so inclined that all will be of the same length, the alternate layers being inclined in opposite directions. When put under strain all wires are equally strained, except the single central wire which acts as a core. This rope bears no resemblance to the ordinary twisted rope. If not larger than No. 8, the wires of each rope can be made continuous from end to end without splicing.

A number of sample ropes were specially prepared by the Trenton Iron Company under the direction of E. G. Spilsbury, M. Am. Soc. C. E., and these ropes were tested at the Watertown Arsenal. The results of these tests will be found in Appendix A. While these tests were of a preliminary character, and the ropes differed in some respects from those which would actually be used in a bridge, they established two important facts: First, that a laid or twisted rope could be made which will develop in the actual rope a strength of more than 180 000 lbs. per square inch of wire, and this with a wire which can be furnished at a reasonable cost; second, that a laid or twisted wire rope of this kind can be depended on in a structure to act uniformly and with a regular modulus of elasticity, the action of the different ropes tested in the latter respect being entirely satisfactory. Twelve ropes were tested in all, of which four were of straight round wires, four twisted round wires, and four twisted wires of the special forms used in a locked rope. The experiments showed the decided superiority of the rope of twisted round wires in all respects except one. The wires were evidently more uniformly strained in this rope than in the straight wire rope, and gave decidedly better results than the peculiar-shaped wires in the locked rope. The only respect in which the twisted rope was inferior to the straight rope was in the modulus of elasticity, which was about 25 000 000 in the

twisted rope, and 28 000 000 in the straight wire rope. The twisted rope was made with a machine already in existence, and with the twist commonly used on similar ropes. For the special use considered, it is probable that the twist could be reduced to one-half or perhaps to one-third that laid by this machine, and that the modulus of elasticity could be raised to about 27 000 000. The modulus of elasticity does not affect the strength of the structure; the only effect of a low modulus is slightly to increase the deflection. In none of the twelve tests, involving twenty-four sockets, did a rope pull out of a socket, but in nearly every instance the fractures occurred in the outer layer of wires and inside the sockets. An examination of the sockets showed a rough shoulder, which undoubtedly had something to do with this fracture. By a modification of the interior shape of the socket, it is probable that this difficulty could be largely removed, and the strength of the ropes increased from 5 to 10 per cent.

Each rope will be fitted into a steel socket at each end 12 ins. long, the diameter of the socket to be twice the diameter of the rope. By adjusting the ropes under strain at the works, and arranging a special machine to trim the under edge of the socket after the rope is fastened into it, it is believed that the length can be so accurately fixed that no further adjustment will be required in the field. If, however, this cannot be done, the arrangement designed permits the employment of fillers under the square shoulders of the sockets, so that the ropes can be adjusted in position.

There will be four cables in the main span of the bridge. There will be four cables in the backstays on each side of the river. There will, therefore, be twelve cables in all, each of which must be fastened at each end.

The method of fastening the cables is shown in Fig. 2. Fifty feet from each end the several ropes, which are compressed compactly together in the body of the cable, begin to separate so that they are 4.9 ins. between centers at the ends, and the successive vertical sets of ropes are 4¼ ins. between centers. On the top of each tower post is placed a steel casting through which all vertical strains are transmitted, and on this casting rests a 20-in. steel pin. On this pin are set up 20 steel plates 2 ins. thick, each plate measuring 10 ft. in the direction of the axis of the cable and weighing 9 255 lbs. The several

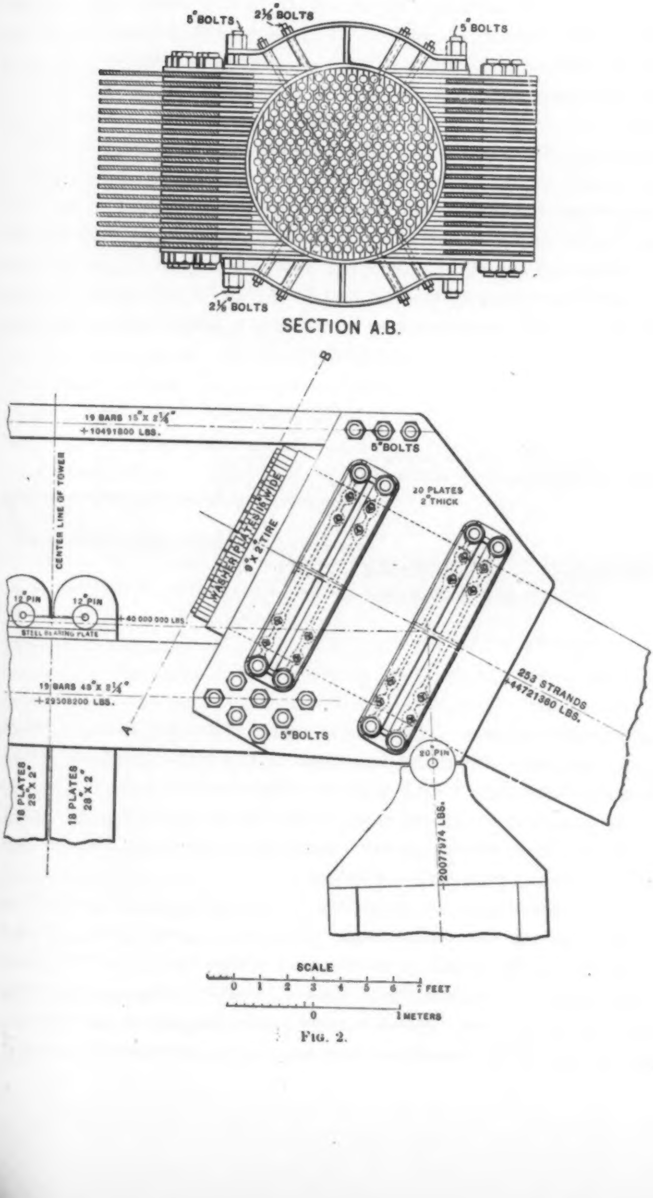

5" BOLTS
2⅛" BOLTS
5" BOLTS
2⅛" BOLTS
SECTION A.B.
19 BARS 15" X 2¼"
+10491800 LBS.
5" BOLTS
CENTER LINE OF TOWER
WASHER PLATES 15" WIDE
9" X 2" TIRE
20 PLATES
2" THICK
12" PIN
12" PIN
40 000 000 LBS.
STEEL BEARING PLATE
19 BARS 48" X 2¼"
+29508200 LBS.
5" BOLTS
253 STRANDS
44721360 LBS.
20" PIN
18 PLATES
28" X 2"
18 PLATES
28" X 2"
20077974 LBS.
SCALE
FEET
METERS

FIG. 2.

ropes of which the main cable is composed, when spread, pass between these several plates, being held in exact position by cheap cast-iron fillers between the ropes. These plates are machined to a true plane surface on the upper edges, and on these are placed a series of washer plates on which the sockets at the ends of the cables bear. These washer plates are 2¼ ins. thick by 16 ins. deep, and the divisions come in line with the centers of the ropes. Each washer plate is bored out for its whole depth on each side with half holes slightly larger than the diameter of the ropes, and for a depth of 10 ins. with half holes of the diameter of the sockets. (See Fig. 3.) Each rope therefore passes through a round hole, one-half of which is bored in each adjacent washer plate and the socket fits into an enlargement of this round hole, bearing on the annular surfaces between the large

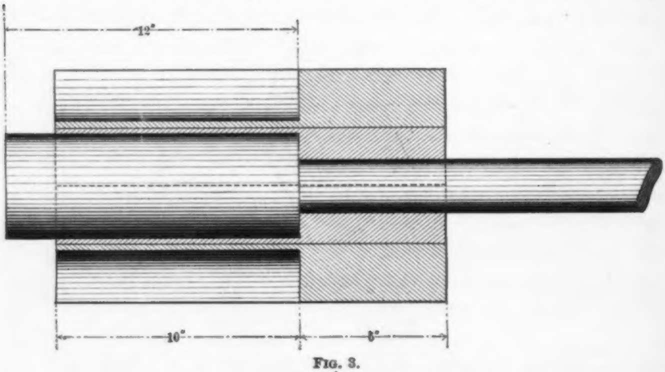


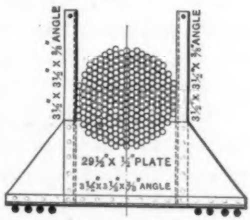
FIG. 3.

and small cylinders. The series of washer plates are bound together by a steel tire shrunk around them. The large plates are bolted together with eight 5-in. bolts and sixteen smaller bolts inclined so as to pass between the ropes, all of these bolts screwing up against heavy cast-steel washers on the outside, the plates being kept at proper distances by the cast-iron fillers.

The entire strain in the cables is transmitted to the large steel plates through the washer plates which bear against them. In the large plates this strain is decomposed into a nearly vertical strain which passes through the 20-in. pin and the steel casting into the post, and a horizontal strain which is taken across the top of the tower to the corresponding backstay connection. For convenience of construc-

tion and erection this horizontal strain is divided between two tension members, the lower one consisting of nineteen bars each 48 x $2\frac{1}{4}$ ins., and the upper of the same number of bars each 15 x $2\frac{1}{4}$ ins., the strain being transmitted to the former by nine 5-in. pins, and to the latter by three 5-in. pins. The full details of this arrangement appear in Fig. 2.

To erect the cables, carrier ropes will be placed above, on which the permanent ropes will be hauled out into place. Eight auxiliary ropes, the general position of which is shown in Fig. 4, will then be run through the unoccupied spaces outside of the washer plates, and on these at suitable intervals a number of iron horses erected. These horses will serve to confine the cable. They are arranged in pairs and braced together, so that they will be stable and will afford room for men to stand and watch the laying of the ropes.



HORSE ON TEMPORARY CABLES DURING ERECTION
Fig. 4.

The side washers, large plates and the nineteen lower ties will be put in position, but the spaces between the plates will be open above. The first washer plate will then be put in place and the first rope unreeled and hauled across the river on the carrier ropes. As soon as it is hauled across, the sockets will be dropped into their places in the washer plates. The first three ropes having been laid in this manner, the next washer plate will be put on as well as the necessary fillers; four other ropes will then be run in the same manner and the third washer plate put on. Five ropes will then be run and the fourth washer plate laid. This process will be continued until all the ropes have been laid and all the washer plates are in position, when the washer plates will be finally consolidated by shrinking on the steel tires. The insertion of the diagonal bolts can begin whenever all the ropes below any one of these bolts are laid. The upper 5-in. bolts will be put in when the laying of the ropes is completed. When this is done the nineteen upper tie bars will be placed, bolted up, and the cables are made. By the use of the horses every rope can be put in its final position as fast as laid, and when each layer of ropes is completed men can be sent over them to make sure that they are properly laid, to paint them and prepare everything in readiness for the next layer.

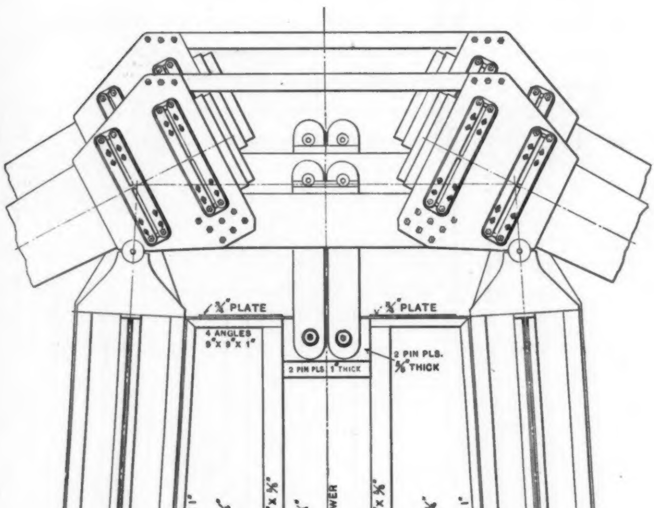
The work is condensed as it goes on, and as soon as the last rope is laid each cable is practically complete. Various special appliances must, of course, be worked out. It may even be necessary to use fillers under the socket bearings and adjust each rope by itself. This description, however, shows the main features of the work. It is believed that at least three ropes can be laid daily, and that a cable can be completed in three working months where everything is organized and ready.

As the small tie bars at the top cannot be put in until the cable is completed, it is necessary to hold the broad lower bars so as to overcome the bending strain due to the center of these bars being below the center of horizontal strain. This is accomplished by the use of 36 bars, 28 x 2 ins. in size, which pass between the 19 bars and anchor those bars down to deep cross girders which connect the separate tower posts, as shown in Fig. 5. These vertical ties serve during erection to take out the bending strain in the lower horizontal tie, and, after erection, to bind the whole arrangement rigidly to the top of the tower.

The backstays are of the same dimensions as the main cables and connected at the top of the towers in precisely the same way, the plates to which the backstays are attached being tied to those to which the main cables are attached. In order to keep the cradled cables of precisely the same length, the outer bearings on top of the tower are lower than the inner bearings, as appears in Fig. 6.

Though the backstays are of the same dimensions as the main cables they carry no weight but themselves and run in approximately straight lines (being deflected from absolutely straight lines by their own weight) to and through the anchorages, each anchorage having two tunnels in it through which the backstays run.

At the lower end the ropes of the backstays are spread between plates in the same manner as at the top of the tower, though the details are different because of the direction in which the strains must be transferred. These details are shown in Fig. 7. The strain in the cable is divided into two equal strains, on lines making equal angles with the axis of the cable, and transferred through 26-in. pins to steel castings which bear on closely cut granite masonry, which is built into the coarser material of the anchorage. The lower pin and castings are put in position before placing the cables, but the upper



¾" PLATE
¾" PLATE
4 ANGLES 9"X 9"X 1"
2 PIN PLS. 1" THICK
2 PIN PLS. ⅝" THICK
4 ANGLES 9"X 9"X 1"
2 PLATES 102"X ¾"
4 SPLICE PLATES 18"X ¾"
2 PLATES 96"X ¾"
CENTER LINE OF TOWER
4 SPLICE PLATES 18"X ¾"
2 PLATES 102"X ¾"
4 ANGLES 9"X 9"X 1"
4 ANGLES 9"X 9"X 1"
¾" PLATE
5"X ½" LATTICE
¾" PLATE
B
PLAN OF TOWERS
ELEVATION FROM A.
0 2 4 6 8 10 12 14 16 18 20 FEET
0 1 2 3 4 METERS

FIG. 5.

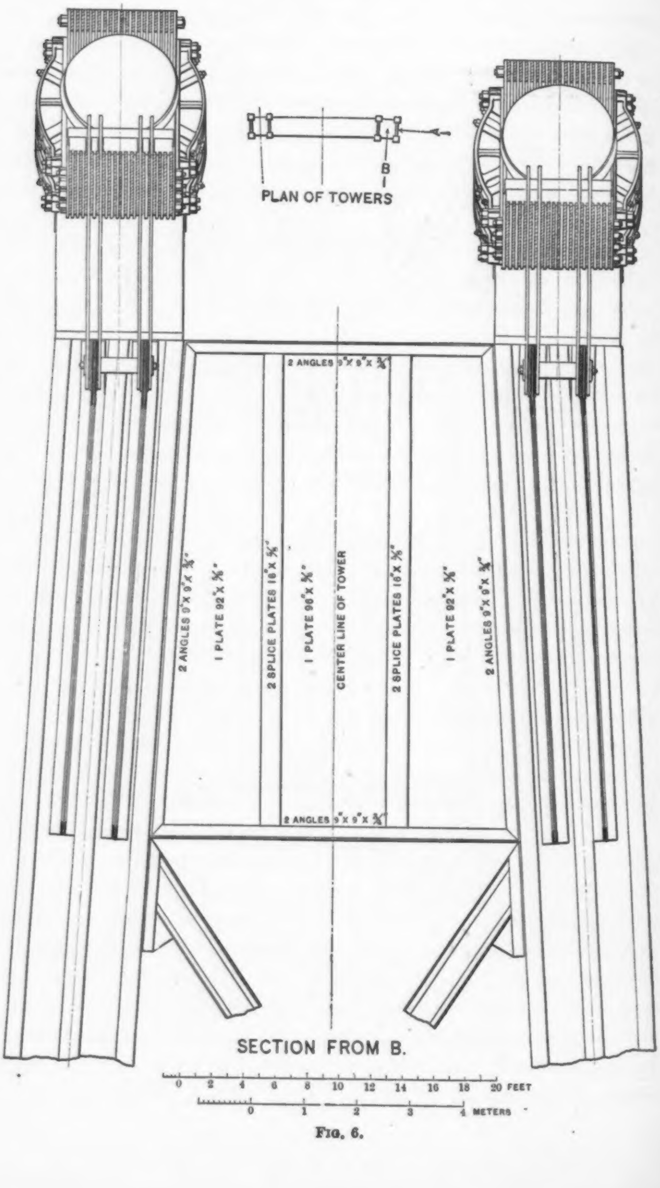

PLAN OF TOWERS
B
2 ANGLES 9"X 9"X 3/4"
2 ANGLES 9"X 9"X 3/4"
I PLATE 92"X 3/4"
2 SPLICE PLATES 16"X 3/4"
I PLATE 96"X 3/4"
CENTER LINE OF TOWER
2 SPLICE PLATES 16"X 3/4"
I PLATE 92"X 3/4"
2 ANGLES 9"X 9"X 3/4"
2 ANGLES 9"X 9"X 3/4"
SECTION FROM B.
0 2 4 6 8 10 12 14 16 18 20 FEET
0 1 2 3 4 METERS

FIG. 6.

one must be omitted so as to slip the ropes between the plates. To compensate for the omission of this pin, nineteen 10 x $2\frac{1}{8}$-in. eye-bars, coupling on a 7-in. pin, pass down a hole sunk into the rock foundation of the anchorage, and are anchored at the bottom of the hole in the manner shown in Fig. 8, these eye-bars serving only a temporary purpose. When the cables are completed, the upper pins and castings are put in position, the granite masonry built up on the castings and

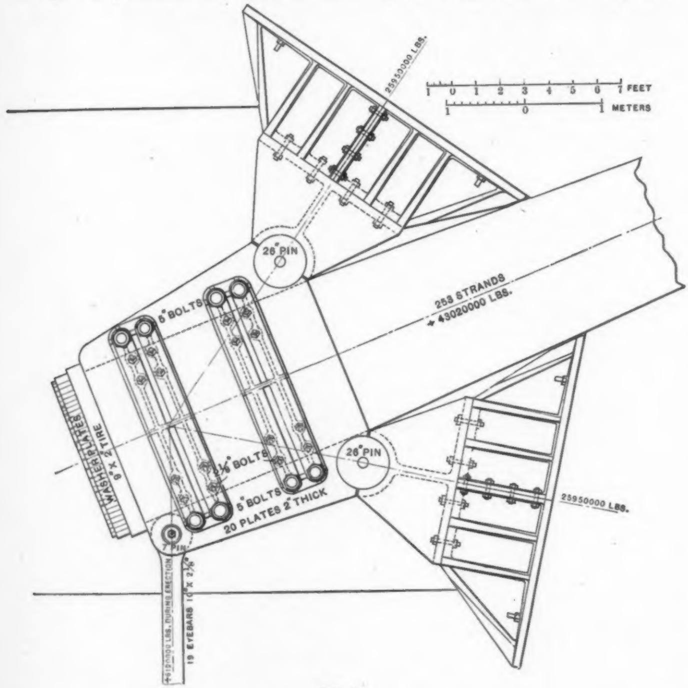


FIG. 7.

the anchorage completed. By this arrangement the cables take hold at once on the anchorage in straight lines without the intervention of eye-bars, and the strains are transmitted in the simplest possible way without the complicated curves commonly used. The cables run through tunnels sufficiently large to walk through conveniently, and the detail at the lower end is in a room about 20 ft. in each direction. The only portions of the work which will not be permanently accessible

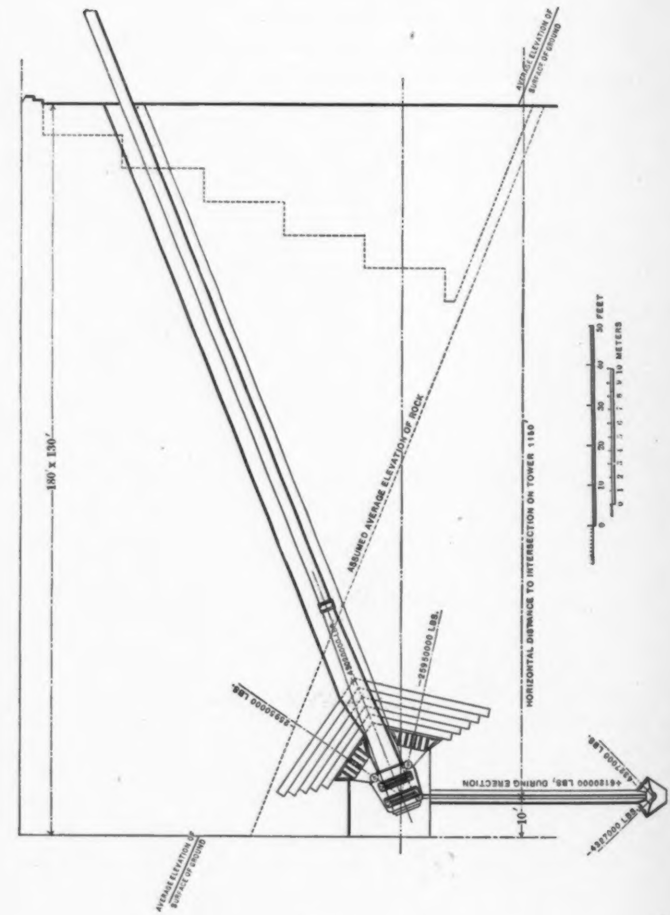

AVERAGE ELEVATION OF SURFACE OF GROUND
180' x 130'
ASSUMED AVERAGE ELEVATION OF ROCK
HORIZONTAL DISTANCE TO INTERSECTION ON TOWER 1180
-25950000 LBS.
+6180000 LBS. DURING ERECTION
10'
50 FEET
10 METERS
AVERAGE ELEVATION OF SURFACE OF GROUND

FIG. 8.

are the eye-bars which form the lower anchorage, and these are only depended on for temporary use.

The calculations referred to above are based on the assumption that the inclination of the cables at the top of the towers is everywhere one vertical to two horizontal; in point of fact, it would be slightly flatter than this, so that the strains in the cables and the reactions on the towers are a little less than has been estimated. The actual inclinations at the top of the towers are according to the following table:

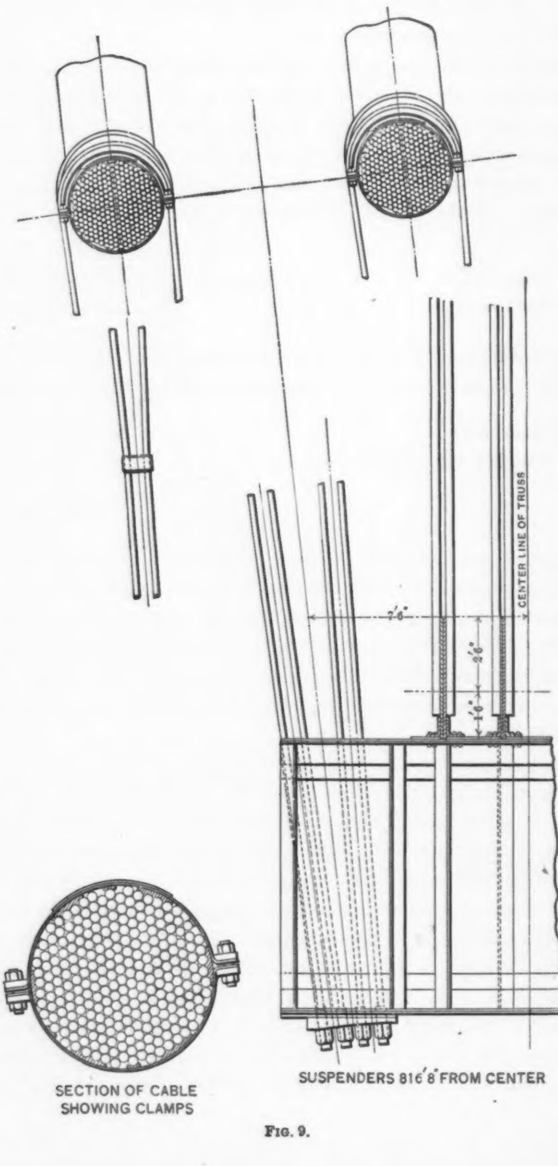
Main cable	1 in 2.196
Backstays	1 in 2.030

The actual length of the cables measured from out to out of the sockets, with an allowance for elongation under strain, is as follows:

Main cable	3 342 feet.
2 backstays, 1 268 ft. each	2 536 "
	5 878 feet.

As there are 253 ropes in each cable and four cables, the total length of rope will be 5 948 536 feet, which, at 10 lbs. per lineal foot, makes a total weight of 59 485 360 lbs. To this must be added the sockets, of which there will be 6 072; each socket will weigh 36 lbs., making the total weight of the sockets 218 592 lbs. The weight, therefore, of the ropes all socketed and ready for erection may be estimated at 59 703 952 lbs.

When the cables are completed the clamps which carry the suspenders will be put on. The form of clamp proposed is shown in Fig. 9, and is quite unlike that commonly used in suspension bridges. The clamp is formed of two steel plates pressed into shape and bound together by eight steel bolts; the lower half is a perfectly plain steel plate, but the upper half has two auxiliary plates riveted on, to hold the saddles which carry the suspenders. By means of cast-iron fillers the irregularities in the cables are filled out, and the whole is then surrounded by a sheet of thin metal about 6 ins. longer than the clamp plates, this thin metal being simply for protection against weather. The clamp-plates are then put on and screwed up tight so that the full friction which can be produced by the bolts is obtained. Two



CENTER LINE OF TRUSS
7'6"
2'6"
1'0"
SECTION OF CABLE
SHOWING CLAMPS
SUSPENDERS 816'8" FROM CENTER

FIG. 9.

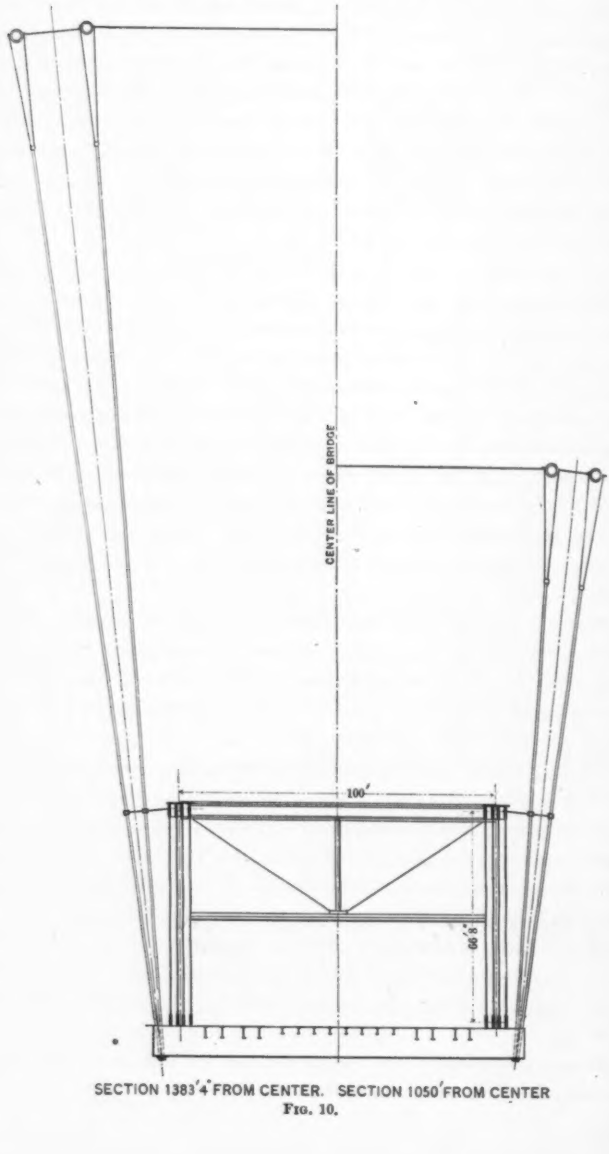
bent saddles of soft metal are then placed on top of the clamp and everything is in readiness to attach the suspenders. Each clamp complete weighs 1 800 lbs., and there are 84 clamps on each cable, making 336 in all. There will also be clamps of different form, but about the same weight at the points where the separation of the ropes begins, 50 ft. from the end of each cable, 24 clamps being required for this purpose. The total number of clamps will, therefore, be 360, the estimated weight of which is 648 000 lbs. The weight of cables and clamps complete will, therefore, be 60 351 952 lbs.

It is thought best not to wrap cables of this size, made of independent ropes, with soft wire as is usually done, but to cover them with a thin layer of some non-conducting substance, which will allow the heat from the sun's rays to reach the metal of the cables no faster than it will be dissipated through the whole volume of the cable. It is premature to say just what the constitution of this non-conducting covering should be; it would probably be finished with a painted canvas surface, and it is not likely to weigh more than 50 lbs. per lineal foot of cable. It would extend from clamp to clamp, covering the projecting ends of the thin metal under the clamps. It must be remembered that it can be easily removed and repaired at any time.

As the cables are first made they will hang in parallel curves between the towers, and as the backstays diverge from the towers there will be a slight transverse tension between the tops of the towers; this, however, is not enough to require any special provision.

The next work, which can be done before the covering is put on the cables, is to cradle them. This cradling will be accomplished by tying the cables on each side together by steel tension bars of varying length, and tying the pairs of cables together by cross-ropes at intervals. These cross-ties and ropes would be attached to every fifth clamp, and the stress in them is comparatively low, amounting to only 90 000 lbs. each, so that ropes of $2\frac{1}{8}$ ins. diameter will be sufficiently large. There will be twelve of these ropes in all, and the total estimated weight of these ropes, together with the ties between cables, is 23 000 lbs.

As soon as the cables are cradled and tied up, everything will be in readiness to attach the suspenders.



SECTION 1383′4″ FROM CENTER. SECTION 1050′ FROM CENTER
Fig. 10.

The total weight of each main cable between vertical intersection points may be taken as follows:

3 325 ft. of cable, at 2 530 lbs. per foot....	8 412 250	lbs.
84 clamps, at 1 800 lbs....................	151 200	"
Cradling-ties............................	5 750	"
	8 569 200	lbs.
3 000 ft. of covering, at 50 lbs............	150 000	"
Total............................	8 719 200	lbs.

This divided by 3 200 gives 2 725 lbs. as the average weight per foot of bridge for each cable and connections, or 10 900 lbs. for the four cables.

As the total weight for which the bridge is proportioned is 50 000 lbs. per lineal foot, or 12 500 lbs. for each cable, the weight which must be carried by the suspenders will be 39 100 lbs. per lineal foot for the four cables or 9 775 lbs. for each cable. There are three suspenders to each 100 ft., so that the weight to be carried by each separate suspender is 325 833 lbs.

The arrangement of the suspenders is shown in Figs. 9, 10 and 11. They are wire ropes of the same character and dimensions that are used in the main cables. The detail selected provides for four suspending ropes at each clamp. Each rope would therefore have to carry 81 458 lbs., equivalent to 27 153 lbs. per square inch, or less than half the stress allowed in the main cables.

There are really but two ropes used at each point, each rope being twice the length of the suspender and fitted at each end into a long socket on which a screw is cut. Each rope passes over the saddle and so forms two suspending ropes; the long sockets pass through washer plates under the floor-beams and are adjusted by nuts under these washers, a detail which might be modified in construction. The suspenders are clamped together about 36 ft. below the cables so as to prevent unnecessary vibration, and where it can conveniently be done the cables will be connected with the stiffening trusses.

The estimated weight of the suspenders, including the sockets, is 1 699 500 lbs., to which may be added 30 500 lbs. to provide for the small vibration connections and extras, making the total weight of suspenders 1 730 000 lbs. or 618 lbs. per foot for 2 800 ft.

The wind strains are transferred to the towers where the stiffening truss passes the towers by cables. There will be sixteen of these cables in all, and the estimated weight of these sixteen cables, including sockets, is 36 650 lbs. The details of this arrangement, which is very simple, will be explained hereafter.

The total weight, therefore, of the rope-work in the bridge, including sockets, will be as follows:

Main cables, as above	60 374 952 lbs.
Suspenders and connections	1 730 000 "
Wind cables	36 650 "
Total	62 141 602 lbs.

These finished ropes can be furnished at the bridge site at a price from 5½ to 6 cents a pound, and the cost of placing them ought not to exceed one-half cent a pound. In these estimates the cost of these ropes has been figured at 7 cents a pound, erected, this including both main cables and all other rope-work, which makes the total cost of cables, suspenders and other similar matters, $4 349 912.

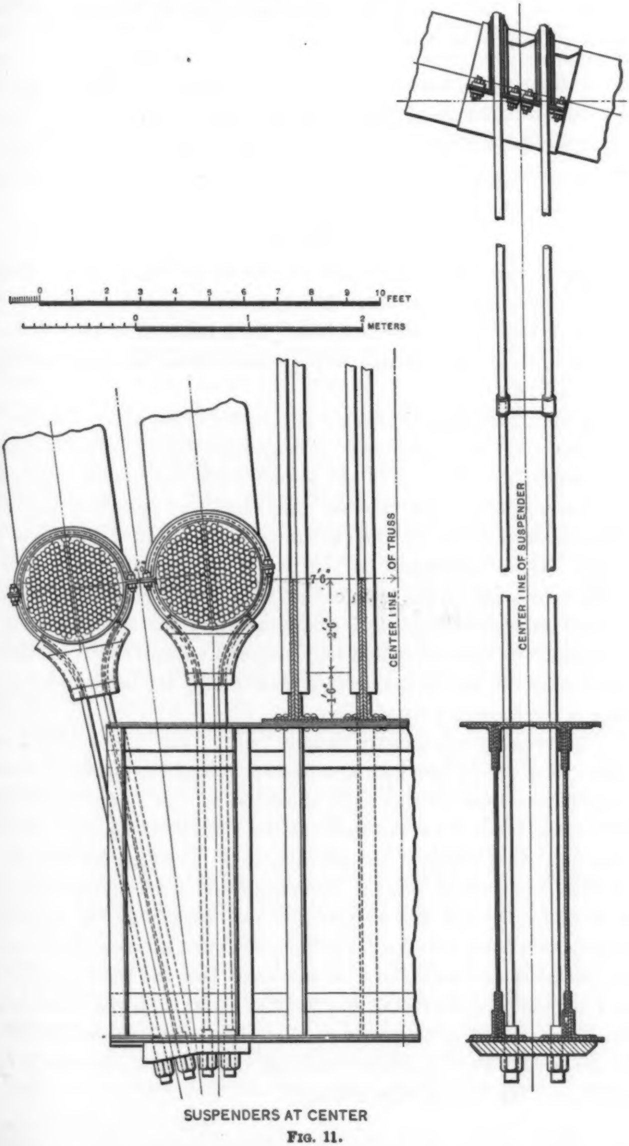
The special details which form the cable connections at the tops of the towers, and in the anchorage, are shown plainly in Figs. 2, 7 and 8. The cable connection on top of each tower-post weighs 454 461 lbs. As there are sixteen posts, the weight of these top connections, including the steel castings, is 7 271 376 lbs.

The connections at the foot of each backstay, including castings and temporary anchorage in the rock, weigh 645 451 lbs., and as there are eight of these, the total weight will be 5 163 608 lbs.

The total weight, therefore, of all material in the special details by which the cables are connected, both at the top of the towers and at the bottom of the anchorage, will be as follows:

Details at top of tower	7 271 376 lbs.
" bottom of anchorage	5 163 608 "
Total	12 434 984 lbs.

This work is generally heavy and simple, but there is in it some nice machine work, and it will probably be expedient to make all the castings of steel; it is, therefore, estimated at 5 cents per pound in position. This makes the cost of these details $621 749.



SUSPENDERS AT CENTER

FIG. 11.

The cost, therefore, of the cables, including all wire rope and connections, will be as follows:

Rope work, etc.	$4 349 912
Connections at towers and anchorage	621 749
22 000 ft. cable covering, at $1	22 000
Total	$4 993 661

Towers.

The towers naturally follow the cables in studying the design. The support of the cables at each end of the main span consists of two towers, which form a double tower. Each tower is of approximately square section, with four corner posts, each battering one in sixteen in both directions.

In designing these towers the special functions which they have to perform must be considered. The arrangement by which the cables are attached to the top of the towers holds the towers absolutely, there being no movable saddles. Any change of length in the backstays must be taken up by a change in the position of the top of the tower. These movements at the top of the tower, combined with changes in length in the main cables, regulate the position of the suspended superstructure. It is important that the towers should be comparatively slender, so that they can bend without overstraining the metal. As the top of the tower is anchored by the backstays, a broad base is not necessary for stability.

The tower is not exactly square, but on the top the north and south sides are in line with the backstays, and the distance between theoretical intersections is 28 ft. on each side. As, however, the outer cables are lower than the inner cables, the actual distance between centers at the bottom of the castings, or at elevation 559.08, will be 29.18 ft. and 29.55 ft. for the north and south sides and 28.42 ft. and 27.58 ft. for the east and west sides of each tower. At the theoretical intersection point between the bottom strut and the posts at elevation 36, the north and south sides of the tower measure 92.94 and 93.8 ft. and the east and west 93.93 and 94.32 ft. Although the tower is not exactly square, it is so nearly so that the irregularity would seldom be observed. The towers, however, are twisted with reference to each other, and this would be manifest.

Strain sheets have been prepared for these towers, and are given in Figs. 12 and 13, these strain sheets showing the calculated results from the 20 000 000 lbs. imposed on the top of each post, and from the weight of the tower itself, beside the strains due to wind. The towers have been proportioned on the basis of a stress of 20 000 lbs. per square inch from weight alone in the posts, no additional provision being made to resist wind or other extraordinary strains, as they will in no event be more than 25% greater than the strains produced by weight alone. The assumption supposes the cables to have an inclination of two horizontal to one vertical. The real strains in these towers are, therefore, from 1% to 10% less than the calculated strains.

The actual motion in the top of the tower after the completion of work, due to changes in the length of the backstays caused by a maximum moving load, on the basis of a modulus of elasticity of 26 000 000, will be $6\frac{7}{8}$ ins., which corresponds to a stress of 2 176 lbs. per square inch in the posts of the tower. This is less than 11% of the 20 000-lb. stress for which the posts of these towers are proportioned; as the towers have been proportioned on the supposition that the angle of the main cable is two horizontal to one vertical, whereas it is 10% greater, and the reaction on the posts 10% less, the stresses in the posts, after allowing for bending, are only 1% more than the calculated strain. The motion at the top of the posts, caused by a change of temperature of 60°, will be less that $5\frac{1}{4}$ ins., corresponding to a stress of 1 450 lbs. per square inch in the posts, which may exist in either direction from a mean.

During erection a much greater motion may be required, especially if no temporary supports are put in for the backstays. A greater motion can be obtained without overstraining the metal, in two ways: by omitting the permanent diagonal bracing and putting in timber bracing, which can be so wedged as to bend the tower; by inserting hydraulic presses in the castings under the posts next to the river, so that the whole tower can be thrown out of plumb.

The section of each post varies from 1 051 sq. ins. at the top to 1 145 sq. ins. at the bottom. Each post is 8 ft. square, and the details of construction are shown in Fig. 14. This post is divided into quarters and made in sections, each 24 ft. long, breaking joints with these sections, so that there would be one horizontal quarter joint every 6 ft. Each quarter section 24 ft. long would weigh less than 12 tons, a

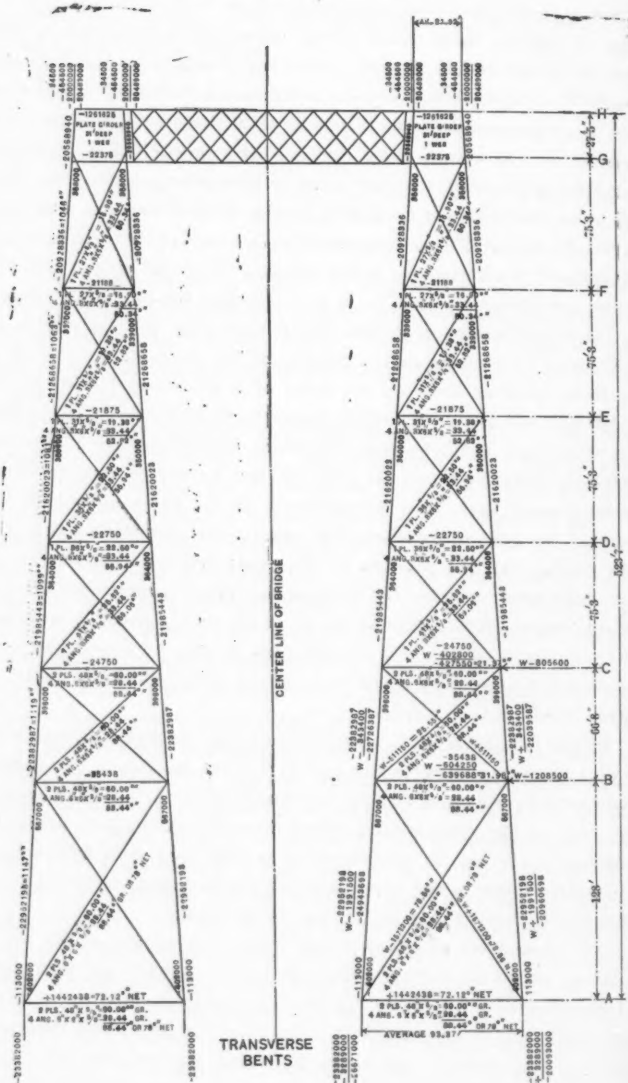

CENTER LINE OF BRIDGE
TRANSVERSE BENTS
H
G
F
E
D
C
B
A
AVERAGE 93.37

Fig. 12.

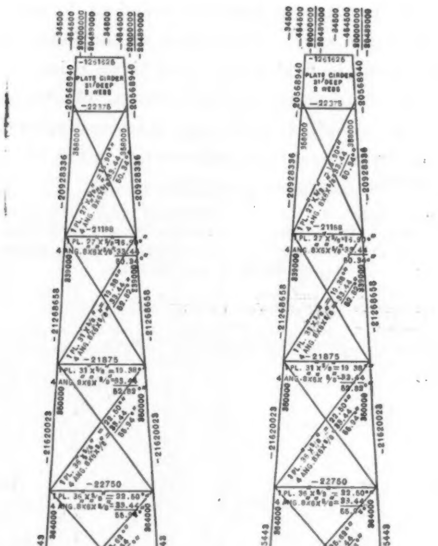


FIG. 13.

weight which can easily be handled. All the heavy riveting around the corners would be shop driven, and it is proposed to use $1\frac{1}{4}$-in. rivets for this portion of the work. The splices at the joints, both vertical and horizontal, would be field driven, $\frac{7}{8}$-in. rivets being used in these places, and could, if necessary, be hand driven. At intervals of 24 ft. diaphragms would be built in each post, these coming opposite one of the joints (as shown in the intermediate section in Fig. 14), the function

BILL FOR ONE-QUARTER POST SECTION.

1 Plate, 48″ x $1\frac{11}{16}$″, 24′ long.	1 Plate, 21″ x $1\frac{1}{16}$″ to $1\frac{5}{8}$″, 24′ long.
1 " $46\frac{5}{16}$″ x $1\frac{11}{16}$″, 24′ long.	1 Angle, 9″ x 9″ x 1″, 24′ long.
1 " 22″ x $1\frac{1}{16}$″ to $1\frac{5}{8}$″, 24′ long.	1 Corner Plate, 25″ x $\frac{3}{4}$″, 24′ long.

2 Splice Plates, 15″ x $\frac{3}{4}$″, 24′ long.

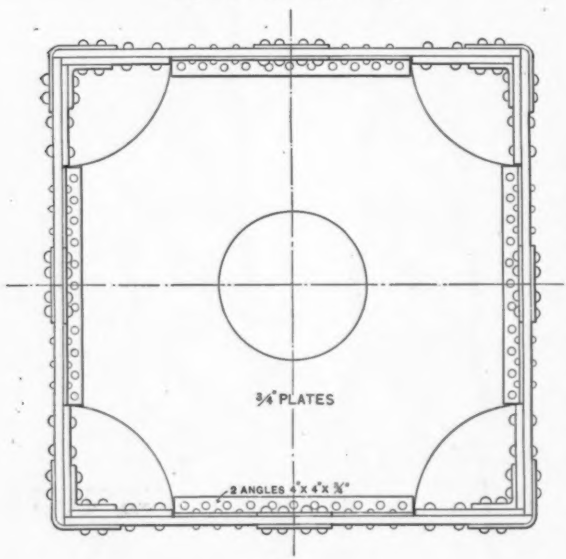


FIG. 14.

of these diaphragms being the same as that of the diaphragms in a bamboo rod. At the top two extra cross webs would be built into the post to support the steel casting.

At the bottom the post would rest on a large casting. For convenience of inspection, a hole is made through the middle of each diaphragm, and a series of ladders would reach from diaphragm to diaphragm, by which inspectors could pass through the whole interior

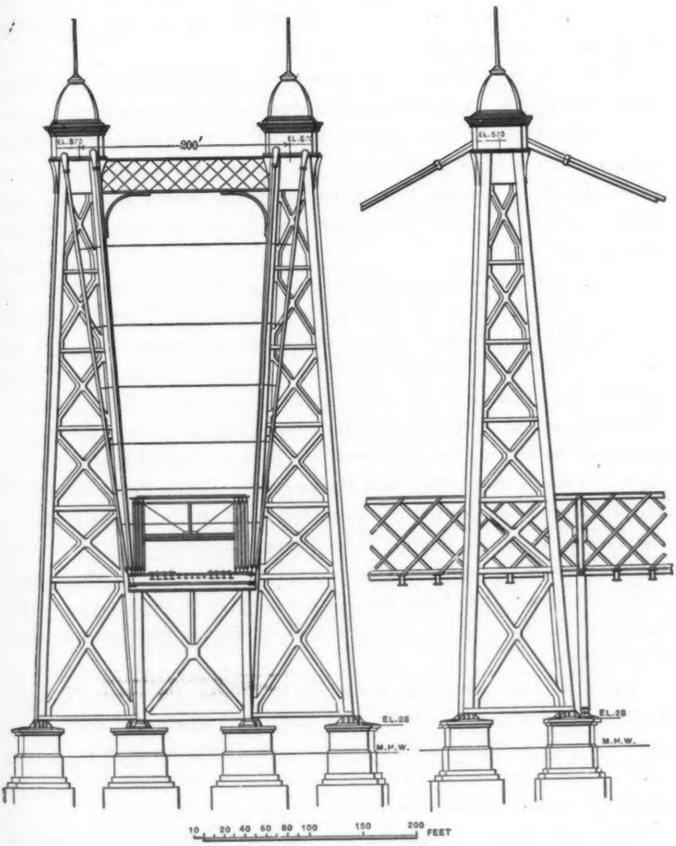

EL.570
200′
EL.570
EL.570
EL.28
M.H.W.
EL.28
M.H.W.
10 20 40 60 80 100 150 200 FEET
10 10 20 30 40 50 METERS

FIG. 15.

of the post, a manhole being placed near the base, through which they could enter. At the bottom each post would be held down by an anchor bolt at each corner, though this is hardly a necessary provision.

These posts have been carefully estimated in detail, the weight of one post being 2 182 000 lbs., or 8 728 000 lbs. for the four, including diaphragms, ladders and everything else.

At the top the four posts are connected by girders 31 ft. deep, there being two girders on each longitudinal side and one on each transverse side. The depth of these girders is fixed by the riveted connection between the double girders and the posts, this connection being necessarily long enough to transfer the whole strain from the vertical bars to the post. These six girders are estimated to weigh 264 000 lbs.

The tower is braced on each side between the four posts, this bracing being divided into six panels, the second panel from the bottom corresponding in height to the depth of the stiffening truss, this arrangement being adopted so that the wind strains can be thrown from the stiffening truss into the tower at the panel points of the bracing. The arrangement of this bracing is given in Fig. 15. Above the stiffening truss the bracing has comparatively little to do, and is made in the form of a single web with broad angles on the edges. From the top of the stiffening truss downward, where the wind strain is to be resisted, the bracing is double webbed and materially heavier. At the bottom the four posts are tied together by heavy riveted ties, which pass outside the posts, but form the bottom member of the bracing. The weight of the bracing for one tower complete has been estimated at 2 389 000 lbs.

The towers are connected at the top by a light lattice truss bridge, which has comparatively little work to do, but which will add to the lateral stability, and be a convenience, both during erection and afterward. This truss bridge complete is estimated to weigh 98 000 lbs., or 49 000 for each single tower.

Each post rests on a large bottom casting, which should be made of steel. This casting would be made in four principal parts bolted together, and should be machined to a true plane on the bottom, as the pressure on the bottom will be 1 000 lbs. per square inch. This casting, together with the anchor bolts, is estimated to weigh 130 000 lbs., or 520 000 lbs. for each tower.

The total weight of the structural portions of the tower will then be as follows:

Posts	8 728 000 lbs.
Girders	264 000 "
Bracing	2 389 000 "
Truss	49 000 "
Castings and anchor bolts	520 000 "
Total	11 950 000 lbs.

As there will be four of these towers, two at each end, the total weight of metal in the towers will be 47 800 000 lbs. This entire work should be open-hearth steel of the quality ordinarily used for structural purposes. The sections are so heavy that the edges would all have to be planed, and it would all be solid drilled work; special machinery would be required for its manufacture, but the order is so large that the cost of this special machinery when averaged over the whole work would be no greater than the wear of ordinary tools. In open competition this work ought to be let for a very low price per pound; if estimated at 4 cents per pound, there results for the cost of the towers $1 912 000.

Fig. 15, showing the general elevation and tower, shows the tower as finished with a room about 50 ft. square on top, surmounted by ornamental work terminating in a flagstaff. This covered room is necessary to house the cable connections, but as the floor of this room will be 560 ft. above the water, or nearly twice as high as any structure now existing in New York City, and higher than any structure in the world except the Eiffel Tower, a considerable revenue could probably be derived by providing elevators and taking visitors to the top of the towers. As the elevators and housings on top of the towers do not form a portion of the structural work, they are not included in the estimate.

Foundations.

The average depth from mean high water to rock at the site of each tower is assumed to be 140 ft. In order to prevent any possible disturbance from expansion and contraction of transverse members at the feet of the metallic towers, it is thought best to rest each post on an entirely independent foundation. There would, therefore, be four independent foundations under each tower, or eight on each side of the

river, making sixteen in all. Of these the two center ones next to the river, on each side, would have to support the reaction of the stiffening truss as well as the weight of the tower post, and they are therefore made larger than the others. The plan of the tower foundations is shown in Fig. 16.

Each separate foundation consists of a masonry pier of granite 58 ft. high, finishing 28 ft. above mean high water, and terminating at the base at an elevation 30 ft. below mean high water. It is proposed to make these piers of an exceptionally good class of masonry and to allow a maximum pressure of 20 tons per square foot on the granite, and of 1 000 lbs. per square inch on the under side of the casting which

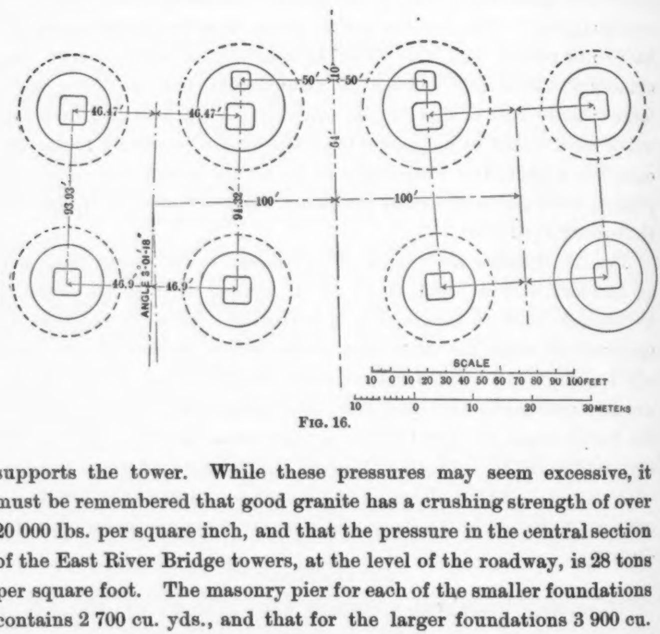


Fig. 16.

supports the tower. While these pressures may seem excessive, it must be remembered that good granite has a crushing strength of over 20 000 lbs. per square inch, and that the pressure in the central section of the East River Bridge towers, at the level of the roadway, is 28 tons per square foot. The masonry pier for each of the smaller foundations contains 2 700 cu. yds., and that for the larger foundations 3 900 cu. yds. Estimating this work at $30 per cubic yard, the masonry for the smaller foundations becomes worth $81 000, and for the larger foundation, $117 000.

Each of the smaller foundations would rest on a cylinder 60 ft. in diameter and 110 ft. high, thus containing 311 000 cu. ft., or, say,

11 500 cu. yds. Each of the larger foundations would rest on a cylinder 70 ft. in diameter, thus containing 423 330 cu. ft., or, say, 15 700 cu. yds. If the cost of this portion of the work is estimated at $20 per cubic yard, the cost of each of the smaller foundations becomes $230 000, and that of each larger foundation $314 000.

The cost of each of the smaller piers complete, including both masonry and foundation, becomes $311 000, and of each of the larger piers, including both masonry and foundations, $431 000. As there are at each end of the bridge six of the smaller piers and two of the larger, the cost of the substructures of each of the double towers will be $2 728 000. The total cost of the foundations for the towers on both sides of the river will then be $5 456 000.

This estimate is believed to be at least $1 000 000 more than the actual probable cost of such work, but this paper is dealing with the general subject of a suspension bridge, and precise estimates of the subaqueous portion really pertain to more specific subjects.

The pressure on the bottom of the foundations after deducting the weight of the material displaced, and estimating the weight of a cubic foot of masonry or foundation at 150 lbs. in air, 87 lbs. in water, or 50 lbs. in mud, is 9.304 tons per square foot for the smaller, and 9.306 tons for the larger.

Anchorages.

The anchorages at each end of the bridge would be divided into two parts, each of which anchors two cables, the position of these anchorages being shown in Fig. 1.

The anchorage has no duty to perform except to provide weight, and may be built of a very cheap class of masonry or of concrete. Any class of work which is entirely free from voids and weighs at least 140 lbs. per cubic foot, or 3 780 lbs. per cubic yard, will answer this purpose. The exposed sides of the anchorages should be faced with a good class of masonry; brick would answer, but granite would be more in keeping with the massiveness of the work. The top will not require a coping, but should be covered with Portland cement concrete or some form of pavement which will keep out water; there is no reason why buildings should not be erected on top of the anchorages.

There will be two tunnels running through each anchorage, each of which should be lined with brick, and be large enough for con-

venient inspection of cables, and perhaps, also, for running a carrier during erection. At the lower end of each cable there will be a room in which the detail connection is placed, and it will probably be expedient to have some kind of a staircase placed in a small shaft by which these two rooms can be reached. The bearing of the castings must be taken on granite masonry of very high quality, the pressure on the bottom of the castings being 1 000 lbs. per square inch, and enough of this masonry has been provided to reduce the pressure on the cheap masonry to 250 lbs. per square inch.

Each anchorage would consist of a single block of masonry, a longitudinal section of which is given in Fig. 8. It is 180 ft. long, 130 ft. wide, and the top finishes at elevation 155, this being the elevation of the rails. The horizontal distance from the theoretical intersection on the top of the tower to the theoretical intersection at the bottom of the anchorage is taken as 1 150 ft. The lower intersection is assumed to be at elevation 60, so that the vertical difference in height between the two intersections is 510 ft.

The volume of material above the theoretical intersection point is 2 223 000 cu. ft., which at 140 lbs. per cubic foot, weighs 311 220 000 lbs. The angle at which the cables take hold of the anchorage is one vertical to 2.524 horizontal. The vertical lift will, therefore, be 31 695 721 lbs. Taking this from the weight of the anchorage, the weight left to resist the horizontal pull is 279 524 279 lbs. Assuming a coefficient of friction of 60 per cent., the frictional resistance of this weight is 167 714 567 lbs. Dividing this last quantity by 80 000 000 lbs., the assumed horizontal strain in cables, gives a factor of stability of 2.09. This is without taking into consideration the strength of the additional anchorage in the rock below, nor including the weight of any buildings or other structures which may be erected on top of the anchorages.

The quantity of masonry in each anchorage above the points of intersection is 2 223 000 cu. ft., or 82 333 cu. yds. As, however, the foundation may be below the assumed level of the intersection, this masonry is estimated as 100 000 cu. yds. The greater part of this would be concrete or a cheap class of masonry, but 1 375 cu. yds. will be high-class granite work. Estimating the whole mass of masonry at $6 per cubic yard, and adding $44 per cubic yard (making a total cost of $50) for the high-class granite work, there results as the cost

of one anchorage $660 500. There will be two anchorages at each end of the bridge costing $1 321 000; the total cost of the anchorages at both ends of the bridge will, therefore, be $2 642 000.

Suspended Superstructure.

The suspended superstructure embraces the floor beams and the stiffening truss, with all the necessary cross-bracing, laterals, etc. The stiffening truss is the principal feature of the whole, and its peculiar function is such that the calculation of the exact strains is a work of extreme difficulty. It is possible, however, in an investigation of strains to bring them without extreme difficulty within limits which they surely will not pass in either direction, and as everything is based on elastic changes in which there must always be a small percentage of irregularity, these results are at least as accurate as the material is uniform, and the error will always be on the safe side.

A stiffening truss with a hinge at the center has the advantage of greater simplicity in the calculations, but the details of the hinge are much more objectionable than any irregularities of strain which might occur, and a continuous stiffening truss without a hinge has been used in this design.

The functions of a stiffening truss may be considered in two ways. It may be regarded simply as a floor stiffener, preventing short local changes; or it may be considered as a complete stiffening truss which distributes the entire moving load with practical uniformity over the whole length of the structure. The former is the usual function performed by the stiffening truss of a long-span highway bridge; the latter is the function which a stiffening truss must perform in a short-span bridge or in a railroad bridge of moderate length.

In the former case the proportioning of the stiffening truss is comparatively simple. The greatest possible distortion of the cables under a moving load must be calculated, the moving load being considered uniform for such distance as will produce the greatest distortion; the amount of deflection which will occur within the limits of this load must then be determined, and the stiffening truss made of such depth that it can deflect this amount without overstraining the metal in the chords; as the deeper the truss, the greater the unit stress in the chords for the same deflection, it follows that the stresses must in this instance be kept down by using a shallow truss. This works

well in highway bridges; it would work equally well in a railroad bridge of such dimensions that the dead load would be so great in proportion to live load that the deflection would not exceed the limit over which trains could conveniently be run at high speeds.

In the case of a railroad bridge in which the dead load is light in proportion to the moving load the stiffening truss must be proportioned on such a basis that it will virtually distribute the whole moving load uniformly over the entire length of span. As the deflection of the stiffening truss has little influence in such calculations, the deeper the stiffening truss, the less will be the stresses in the chords.

In the present case the dead load is so great in proportion to the moving load that the distortion of the cables will be comparatively small, even under the passage of trains; it will, however, be so great that if the stiffening truss performed no other functions than that of a floor stiffener, the deflection might disturb the rapid passage of trains. In this case, the proper method of proportioning a stiffening truss is, to decide first what deflection will be allowable under maximum conditions; then to determine what depth of stiffening truss will correspond to the maximum unit stresses which it is considered wise to adopt; then to determine the amount of difference in load which will be required to create a corresponding distortion in the cables; by deducting this difference from the total moving load, the amount of moving load is determined which the stiffening truss must distribute. On this basis a stiffening truss may be proportioned, though the exact strains must be a matter of subsequent computation.

The condition of loading which will cause the greatest deflection in the loaded portion of the stiffening truss will occur when the maximum moving load covers one-half of the 2 800 suspended feet of stiffening truss, occupying 1 400 ft. on either side of the center; this is also the case in which all calculations are most simple. A limit of deflection of one four-hundredths of the half span, or 3½ ft. in 1 400 ft., corresponds to a 1% grade at each extreme end of the deflection, and has been selected as the limit in this case.

The moving load which the cables are capable of carrying is equivalent to 11 000 lbs. per lineal foot over the entire structure, and it is assumed that this load is distributed over the equivalent of six railroad tracks, corresponding to 1 833 lbs. per foot

of track. In estimating the effects of an unequal load a weight of 12 000 lbs. per lineal foot is taken in accordance with the provisions stated at the beginning of this paper. While it may be considered that the load per foot on one-half or one-third of a span ought to be more than 10% greater than the load per foot on the whole span, it must be remembered that the peculiar conditions of this bridge are such that it is only under very rare conditions that any considerable portion of the moving load must be distributed by the stiffening truss. In fact two passenger trains could cross this bridge side by side without disturbing the position of the cables beyond the limits of deflection which are permissible; it is only when the load exceeds that of two maximum passenger trains that the stiffening truss has any duties to perform beyond that of a floor stiffener.

The excess load required to produce a distortion of 3.5 ft. at the quarter on a 2 800-ft. span with a versed sine of 310 ft. (which corresponds to the design), will be 9.424% of the load on the unloaded portion. Taking the dead load at 39 000 lbs. per lineal foot, 9.424% of this is 3 675 lbs.; deducting this from 12 000 lbs., there remains 8 325 lbs. as the weight per foot to be distributed by the stiffening trusses, or 4 162 lbs. for each truss.

By the use of nickel steel, as hereafter explained, considerably greater strains may be used in the stiffening truss than would be considered good practice with ordinary structural steel; a stress of 17 000 lbs. per square inch has therefore been selected as the limit. This having been assumed, the depth of the stiffening truss may be calculated by the following formula:

$$d = \frac{5\, S\, l^2}{24\, E\, v_o}$$

in which

S = stress per square inch = 17 000
E = modulus of elasticity = 30 000 000
v_o = deflection = 3.5
l = span = 1 400

Solving this quotation, $d = 66.11$.

For convenience the depth has been made 66 ft. 8 ins., equal to $200 \div 3$.

The required section of the chords of the stiffening truss will then be determined by the following formula:

$$a = \frac{\frac{w}{2} l^2}{8\, d\, s}$$

in which

$$\begin{aligned} d &= 200 \div 3 \\ &= 1\,400 \\ s &= 17\,000 \\ w &= 4\,162 \end{aligned}$$

Solving this equation, $a = 450$.

In the design the chords of the stiffening truss have a gross section of 600 sq. ins. and a net section of 560 sq. ins.; the net section is 24% greater and the gross section 33% greater than the approximate calculation requires; it is expedient, however, to provide an excess of metal above that required by these advance calculations. The gross section is the one to be used in calculating deflections. Should the modulus of elasticity assumed (30 000 000) be criticised as being too high, it must be noted that this modulus is applied to the calculated section of the chord, whereas the actual section would be materially increased by splices and connection plates, so that a modulus of 27 500 000 in the metal would correspond to at least 30 000 000 in the calculated section.

This is a simple solution and gives data to start from, but while considering the effect of the distorted cables in carrying unequal loading, it does not consider the bending effects on the stiffening truss due to the deflection of the cables as a whole, all of which must be taken into consideration in the final calculations. In these final calculations it is necessary to assume certain sections of cables and stiffening truss chords as well as loads. It is not possible to make a strain sheet in advance and then proportion the sections in accordance with the strains. Everything is determined by deflections, and deflections are themselves determined by sections.

A careful investigation of the theory of the stiffening truss has been made on entirely independent lines by Mr. Charles S. Peirce. By the use of the formulas which this investigation has developed, the strains in the stiffening truss have been calculated for three different conditions. In the first of these a moving load of one-third the length

of the span, 3 100 ft., is assumed to occupy the first third of the 3 100 ft. In the second supposition it is assumed to occupy the second and third sixths of the 3 100 ft., the end of the load being at the center. In the third supposition it is supposed to occupy the central third of the 3 100 ft. The moments developed by these calculations are a satisfactory confirmation of the capacity of the stiffening truss.

In this connection it is interesting to compare the duty which is to be performed with that on the East River Bridge. The stiffening

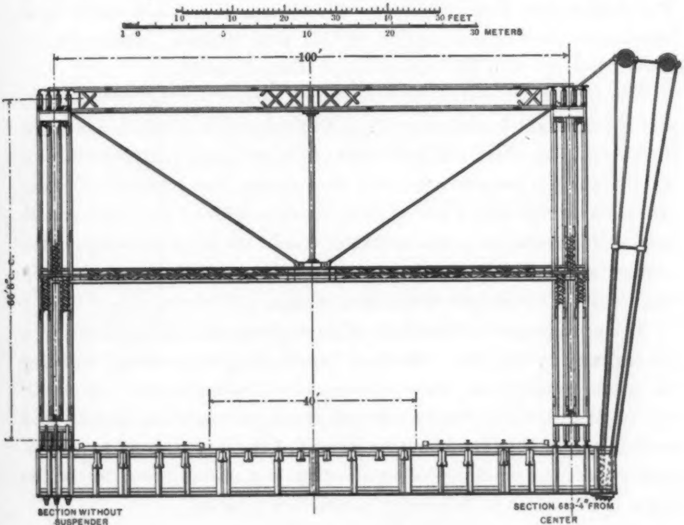


FIG. 17.

truss of the East River Bridge is simply a floor stiffener; both chords are cut at the centers of both the main and the side spans, having sliding joints which can transfer no bending strains whatever and a comparatively small amount of shear. The trains which run over this bridge weigh about 100 tons each, or half a ton per lineal foot, about two-thirds the weight of a first-class passenger train, and they are about half as long as an ordinary passenger train; and yet, in spite of the lightness and theoretical inadequacy of the stiffening truss, the action of the bridge under these trains is entirely satisfactory.

In all these calculations no account has been taken of the stiffness of the cables themselves, which are more than 3 ft. in diameter, of the chords of the stiffening truss, which are 4 ft. deep, nor of the stringers of the floor system, all of which would have decided influences of their own.

The two stiffening trusses designed are each 66.67 ft. deep between centers of gravity, or 70 ft. over all. They are placed 100 ft. between centers. There is a stiff riveted lateral bracing between the top chords and a transverse bracing, as shown in Fig. 17, at every panel point. The floor system is entirely below the bottom chord and the bottom laterals are built in as a portion of this floor system. The webs are riveted lattices with four independent lines of bracing.

The suspended portion of the truss is carried by the floor beams, and as its weight exceeds the amount of moving load which has to be distributed, its action really amounts to varying the portion of its own weight which is transferred to the floor beams, there never being any conditions under which any portion of the weight of the floor system has to be transferred to the stiffening truss. Beyond the limits of the suspenders the floor beams are hung from the stiffening truss to which they transfer the weight of the floor system.

The arrangement of the floor system as worked up is shown in a general way in Fig. 18. The floor beams are strong enough to carry the whole weight from truss to truss, thus leaving a clear space for the whole distance. The two double-track railroads are placed next to the trusses, thus reducing the weight of the floor beams to a minimum, while the possible deflection of one end of the beam below the other is found not to be enough to produce trouble.

There will be eight railroad stringers in each panel and eight lighter stringers which carry the roadway or the rapid transit tracks. Each railroad stringer is proportioned for a total load of 4 230 lbs. per foot of stringer, the strains in the extreme fiber being 9 025 lbs. per square inch of gross section, and weighs 6 300 lbs. Each light stringer is proportioned for a total load of 1 000 lbs. per foot of stringer, the strain in the extreme fiber being 9 620 lbs. per square inch of gross section, and weighs 2 540 lbs.

The floor beams are proportioned to carry a weight of 45 667 lbs. at each single railroad stringer connection, and for a load of 100 lbs. per square foot on the 40 ft. between railroad tracks, besides a weight of

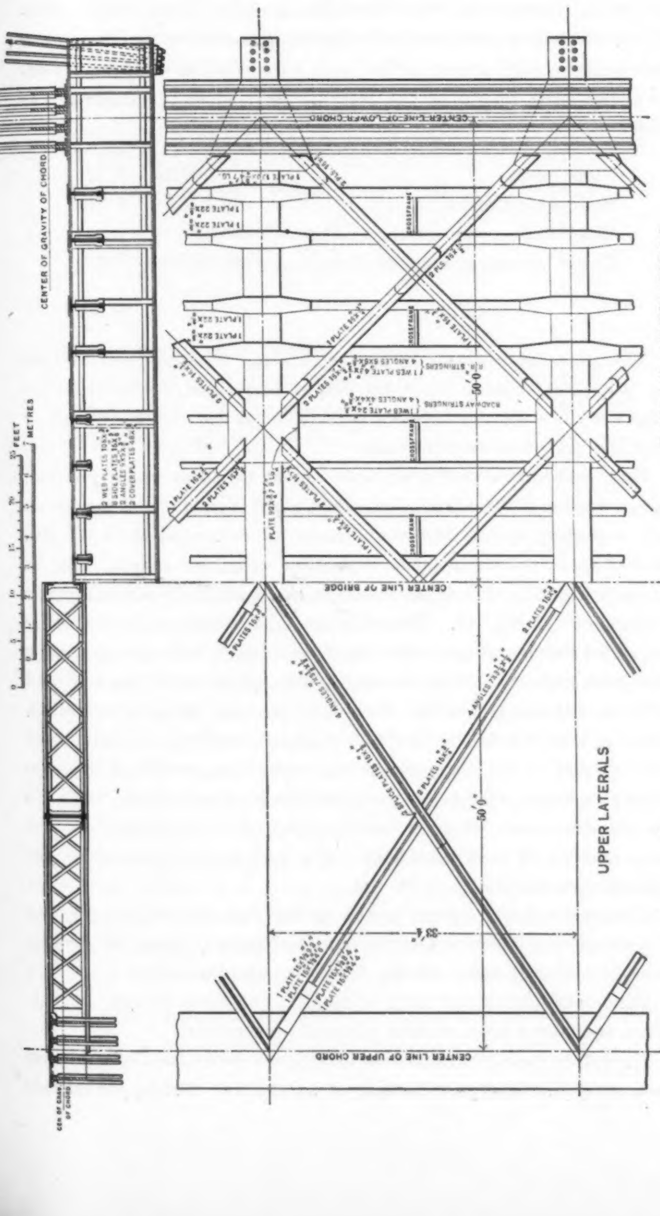

CENTER OF GRAVITY OF CHORD
CENTER LINE OF LOWER CHORD
FLOOR AND LOWER LATERALS
CENTER LINE OF BRIDGE
UPPER LATERALS
CENTER LINE OF UPPER CHORD
FEET
METRES

Fig. 18.

280 000 lbs. at each end from the stiffening truss. Under these conditions the strain in extreme fibers of gross section is 12 938 lbs. Each floor beam is really a box girder with two webs, and weighs 179 000 lbs. The total weight of the metallic floor system is therefore as follows:

	Per panel.	Per foot.
Floor beams	179 000 lbs.	5 370 lbs.
Railroad stringers	50 400 "	1 512 "
Rapid transit stringers	20 320 "	610 "
Lower laterals and connections	24 880 "	746 "
Total	274 600 "	8 238 "

The upper laterals and transverse bracing together weigh 48 000 lbs. per panel, making the total weight suspended, exclusive of the weight of the stiffening trusses themselves, 322 600 lbs. per panel, or 9 678 lbs. per lineal foot of bridge.

Each chord of the stiffening truss has, as has been already stated, a gross section of 600 sq. ins., and the weight of each chord per lineal foot, including splices and connections, is estimated at 11 100 lbs. Each of these chords is made with four webs, the details being as shown in Fig. 17. The arrangement of the webs of the stiffening truss is also shown in Fig. 17. The webs are made uniform for the entire suspended portion of the stiffening truss. Each web member has a total gross section of 53 sq. ins. and a net section of 41 sq. ins., and under the extreme conditions which have been specified above, namely, a moving load of 8 325 lbs. per foot of bridge, exclusive of that which is distributed by the distortion of the cable, these members are subjected to a strain of 7 855 lbs. per square inch of net section, and 6 373 lbs. per square inch of gross section, these strains occurring in the same member in both directions. The web as so designed weighs 3 240 lbs. per lineal foot.

The dead weight of floor, including ties, rails and other work, has been assumed at 3 000 lbs. per lineal foot of bridge, being 500 lbs. for each railroad track and 1 000 lbs. for the intermediate 40 ft.

The lower laterals act only in tension, and their weight, as estimated, includes a large amount of detail connections.

The top laterals and transverse bracing are determined by minimum sections for the most part instead of by strains. The upper laterals

weigh 750 lbs. per lineal foot, and the transverse bracing 690 lbs. The total weight, therefore, of the suspended superstructure per lineal foot may be taken as follows:

Metallic floor as above	8 238	lbs.
Stiffening truss chords	11 100	"
Stiffening truss webs	3 240	"
Upper laterals and cross-bracing	1 440	"
Total metal work	24 018	"
Tracks and flooring	3 000	"
Total	27 018	"

The total dead load is therefore as follows:

Cables and connections (page 493)	10 900	lbs.
Suspenders and connections (page 493)	618	"
Suspended superstructure	27 018	"
	38 536	"
Add for telegraph line and sundries	464	"
Total	39 000	"

This leaves 11 000 lbs. of the total 50 000 lbs. available for moving load, as already stated, which may fairly be considered a margin of 2 000 lbs. over anything that is likely ever to occur.

The total length of the stiffening truss from out to out, including the 500-ft. spans at each end, is 4 100 ft. Assuming chords and floor system to be uniform throughout, the weight of this 4 100 ft., taken at 24 000 lbs. per lineal foot, will be 98 400 000 lbs.

This, however, includes the weight of the heavy floor beams within the suspended length, while there are four floor beams entirely omitted at the supporting points and thirty-six floor beams which are themselves suspended from the chords. Neglecting one end floor beam, as the estimate has been made on a basis per lineal foot, and assuming that the floor beams hung from the chords are 30 000 lbs. lighter than the others, there is a deduction on this account as follows:

3 floor beams at 179 000 lbs	537 000	lbs.
36 floor beams reduced, at 30 000	1 080 000	"
	1 617 000	"

On the other hand, the webs near the supporting bents will have to be reinforced, which can be done by making the members of greater width than elsewhere. As there are no reverse strains here, it is thought right to fix the limit of stress at 20 000 lbs. per square inch of net section. On this basis the additional metal required in the webs is 3 418 000 lbs.

To this must be added the weight of four vertical posts over the rocking bents, and four vertical posts and two portals at the ends of the continuous superstructure.

The weight of the entire suspended superstructure will then be as follows:

4 100 feet (page 515)	98 400 000 lbs.
Additional metal in web	3 418 000 "
Vertical posts over rocking bents	545 000 "
End posts	467 000 "
Portals	140 000 "
Total	102 970 000 "
Deduct for floor beams	1 620 000 "
	101 350 000 "

At the center of the bridge the cables are so little above the floor beams that the stiffening truss must be considered as fastened to the cables longitudinally at this point, a condition which is assumed in the refined calculations of strains; as it is continuous it must be free to move longitudinally at all other points, and especially at the ends. As the continuous truss is 4 100 feet long, a motion due to the effect of temperature on 2 050 ft. must be provided for at each end, this being 0.82 ft. for 60° of temperature; a possible motion of 1.64 ft. must therefore be provided at each end of the stiffening truss, or 0.82 ft. from either side of a mean. This motion is too great to be accommodated by roller bearings of the ordinary kind, and the design places the stiffening truss on rocking bents, which are shown in Figs. 15 and 19. The possible reaction at points 3 100 ft. apart is assumed to be 10 736-000 lbs., and this is taken on vertical posts, the pair of posts under the two trusses being braced together, thus forming a rocking bent, which is supported on two of the cylinders which form the tower foundations. These special cylinders have therefore to sustain this

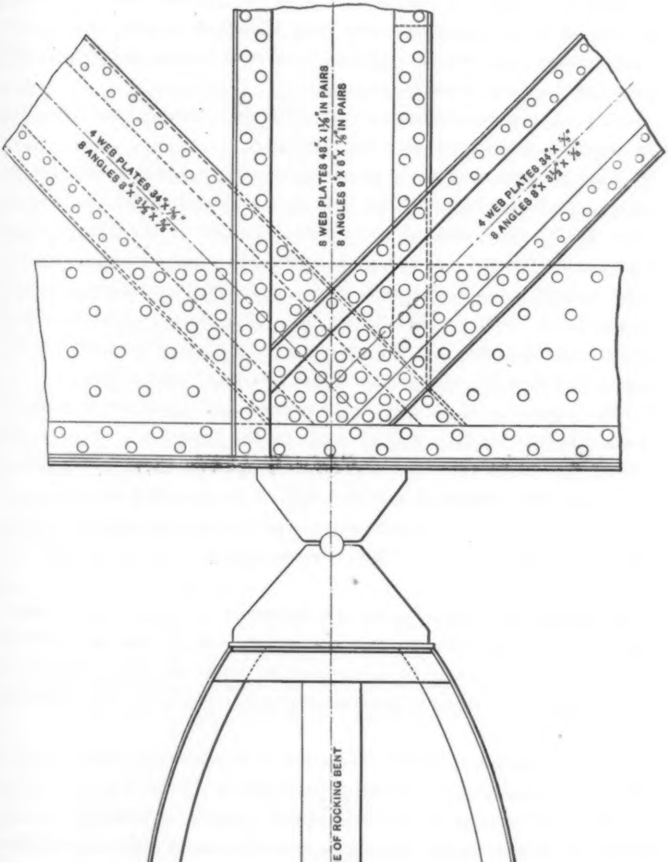

4 WEB PLATES 34" X ⅞"
8 ANGLES 8" X 3½" X ⅞"
8 WEB PLATES 48" X 1⅛" IN PAIRS
8 ANGLES 9" X 6" X ⅞" IN PAIRS
4 WEB PLATES 34" X ⅞"
8 ANGLES 8" X 3½" X ⅞"
CENTER LINE OF ROCKING BENT

Fig. 19.

weight in addition to the weight received from the tower, and for this purpose their size has been increased, as has already been stated, and the center of the foundation has been placed at a point between the two bearings, the distance of which from each bearing is inversely proportional to the respective reactions.

At each end the stiffening truss rests on a rocking bent of smaller dimensions, which rests on a masonry pier.

Each rocking bent at the principal point of support is estimated to weigh, complete, 1 005 000 lbs., and each rocking bent at the extreme ends 480 000 lbs., making, as the total weight of the four rocking bents, 2 970 000 lbs. If to this is added the weight given above, the total weight of the metal work in the suspended superstructure and connections becomes 104 320 000 lbs. This work would all be of structural steel, and in it the strains will nowhere exceed 20 000 lbs. per square inch of gross section under actions of load alone.

The strains in the chords are reversed, and these are further increased by the bending which occurs in the truss under the rise and fall of the cables from the effects of temperature and by the effect of wind. The estimated rise and fall at the center from changes of temperature is 3.3 ft., which, calculated on the same formula as used previously and taking $l = 3\,100$, corresponds to a strain of 3 330 lbs. per square inch.

The bending strain due to the deflection of cables under weight has not been considered in this connection, but in the more refined calculations is considered in connection with all other changes of shape caused by weight in determining the moments on the stiffening truss.

The strains are subject to reversal, and represent, including the effects of temperature, a possible variation of 40 000 lbs. per square inch between extreme positive and negative strains, or 20 000 lbs. in each direction ; this is higher than it is deemed wise to place on ordinary structural steel, and requires a material which, while possessing the toughness of the soft steel preferred for structural purposes, has the strength and high elastic limit of the harder steels. Five years ago such a material would have been considered impossible ; it may now be found in nickel steel containing about 3½% of nickel, a material which will have an elastic limit of about 60 000 pounds per square inch and can be subjected to the reverse strains just referred

to, and under extreme occasional conditions could be worked to at least 40 000 lbs. per square inch without injury. As nickel steel is a comparatively new article, made by a few manufacturers, though it has been adopted to a very great extent by the United States Government in its naval work, it is difficult to learn just what the additional cost ought to be ; apparently it is worth, on the basis of cost, about three-quarters of a cent more per pound than ordinary structural steel, but it has been estimated as costing 2 cents per pound extra, this representing additional mill and shop work, though the latter is very little.

The modulus of elasticity of nickel steel is practically the same as that of ordinary structural steel, and it is proposed to use it only for the principal members of the chords for a length of 2 000 ft. at the center of the bridge, where reversals and wind strains are large. The rivets, splice plates, etc., need not be of nickel steel. The weight of nickel steel in each chord may therefore be taken at 2 000 lbs. per foot, or 8 000 pounds for the four chords, or 16 000 000 lbs. for the 2 000 ft.

The work in the stiffening truss is of a very uniform character, and, considering its great weight, ought to be obtained at a very moderate price per pound ; it is estimated at 4 cents per pound, with an extra allowance for nickel steel.

The total cost of the 4 100-ft. stiffening truss, supporting bents, etc., may therefore be taken as follows:

104 320 000 lbs. at 4 cents....................	$4 172 800
16 000 000 " nickel steel, at 2 cents extra...	320 000
Total	$4 492 800

Shore Piers.

To sustain the ends of the stiffening truss two additional piers will be required. These piers should be founded on rock, but would be piers of ordinary dimensions, and, though large, would present no special difficulties of construction. On the plans, Fig. 1, they are shown with the masonry finishing at an elevation 60 ft. above mean high water, which is probably higher than necessary, and the depth to rock is assumed to be 80 ft. below mean high water. The piers are assumed to be 20 ft. wide and 120 ft. long on top, the masonry to start

at the water level and to be founded on a caisson and surmounted by a timber crib filled with concrete, the whole foundation being 35 ft. wide, 135 ft. long and assumed as 80 ft. high. The cost of these piers is estimated as follows:

6 000 cu. yds. masonry at $25	$150 000
378 000 cu. ft. foundation at 60 cents	226 800
Total	$376 800

The cost of the two piers, one at each end, would then be $753 600.

Wind Pressure.

The wind surface per lineal foot presented by one-half of one web, the lower chord and the floor system is 11.35 sq. ft., and the wind surface presented by the upper half of the web and the upper chord is 7.77 sq. ft. As the trusses are 100 ft. apart, the area of the trusses should be doubled, but the floor comes so near to being solid that it need not be doubled. The total surface presented to the wind which must be resisted by the top laterals is therefore 15.54 sq. ft. per lineal foot, and the total surface presented to the wind which must be resisted by the bottom laterals is 19.12 sq. ft. per lineal foot. To the latter should be added the area of a passing train, which is equivalent to 8 ft. above the bottom chord, thus making the total wind surface to be provided for 27.12 sq. ft. per lineal foot. On a basis of 30 lbs. per square foot the total wind pressure to be resisted is—

Top lateral system	466 lbs.
Bottom lateral system	814 "
Total	1 280 "

For the calculations, these figures have been slightly varied, and the top laterals are proportioned to resist a wind pressure of 500 lbs. per lineal foot and the bottom laterals a wind pressure of 750 lbs. per lineal foot.

There is no probability that anything like these wind strains will ever be reached over the whole length of the span, though considerably greater pressures may occur for limited lengths. To reduce these amounts, however, would be a departure from established practice.

The wind pressure would be transferred to the towers where the stiffening truss passes the towers, by horizontal cables, these cables reaching from each chord to the outer posts of the towers, the cables clearing the inner posts and being long enough to provide for the longitudinal motion of the trusses without overstraining. These horizontal cables would be tightened under strain so that they would always stiffen the trusses. A portion of the wind strain would undoubtedly be taken by the transverse bracing of the rocking bent. Furthermore, the continuity of the truss beyond the rocking bent would reduce the equivalent length of the central span. In calculations this reduction has been assumed to be about 50 ft. at each end, though this is undoubtedly much less than it would really be. On this basis the bending strain produced by wind in the bottom chords will be—

$$\frac{3\,000^2 \times 750}{8 \times 100} = 8\,437\,500$$

This corresponds to about 14 000 lbs. per square inch on the 600 sq. ins. of the bottom chord, and gives a deflection calculated as above of 8.75 ft. Should this occur when there is a maximum strain in the chords from the passage of trains, a condition which would probably not take place more than once in a century, the chords might possibly be strained to 34 000 lbs. per square inch. With nickel steel this is perfectly safe.

In these calculations it is assumed that the chords of the stiffening truss are the only longitudinal members, which is by no means correct, as the sixteen stringers will act as auxiliary chords in the wind system.

There is also another element which materially reduces the effect of wind. To produce the above-mentioned strains in the chords, the whole suspended superstructure must move laterally 8.75 ft. This involves swinging the main cradled cables and raising the center of gravity of the suspended superstructure, a lateral movement of 8.75 ft. corresponding to a lift of 0.075 ft. or 1 vertical to 117 horizontal. As the suspended superstructure weighs 27 000 lbs. per lineal foot, this will require a horizontal force of 230 lbs., so that before this deflection can occur the actual wind pressure must be about 1 000 lbs. per lineal foot on the bottom chord.

Rivet Strains.

While plans showing the details of riveting have only been prepared as a study, it has been necessary to form some basis on which they should be proportioned, especially as owing to the magnitude of the structure and the large relative dead loads, the unit strains vary from those ordinarily used. The simple rule has been followed that the bearing stress on each rivet should be considered equal to the stress allowed in the gross section of the member, and the shearing stress should be limited to one-half the stress allowed in the gross section of the member; this may be expressed differently by stating that the bearing surface of the rivets should not be less than the gross section of the member, and the shearing section of the rivets should be double the section of the member. Where nickel steel is used, the number of rivets is increased one-half, the bearing surface of the rivets being made 50% greater than the gross section of the member and the shearing section of the rivets three times the gross section of the member.

Erection.

The erection of the stiffening truss and suspended superstructure is a comparatively simple thing. The back spans, 500 ft. each, would be erected on falsework in the usual manner, which could be put in without difficulty as it is in a protected position back of the pierhead lines. The projections from the rocking bents to the suspenders would be built out as cantilevers, all of which could be done without trouble.

The suspended superstructure proper would be handled in a different way. The floor beams would be put in position first; they would be brought to the bridge site on barges and raised into position, each beam being hung from the suspenders as fast as raised. The stringers would be put in and riveted up as the floor beams are erected, so that when all the floor beams are up, a reasonably stiff floor would be ready to work on; this portion of the work could be done very rapidly, as each of the 84 floor beams could be handled independently.

When the floor beams are in place and the floor system riveted up it could be covered with planks and form a working platform. The bottom chords of the stiffening truss would then be put in place and riveted up, the horizontal rivets being driven by power, and the vertical rivets, which are of less importance, by hand. The only matter which would require special attention would be to see that a uniform distribution of weight was kept at all times, and this is a

matter of discipline rather than of difficulty. As soon as the bottom chord is riveted the lower half of the webs would be erected and this would be followed with the upper half, after which the top chords would be put on and the top lateral system erected. The broad floor would form a platform on which any desirable system of travelers could be run and the opportunities for work would be as good as in a shop, except that there would be no roof.

The total weight of suspended superstructure which must be erected in suspension is about 34 000 tons. The speed with which it could be handled would depend entirely on the number of men and the amount of plant employed.

Estimate.

The work has been described in the manner in which the design has taken shape, and the cost of each separate portion has been estimated in connection with this description. In execution the work would necessarily be differently divided and may properly be grouped under the respective heads of substructure and superstructure.

Under these heads the cost may be stated as follows:

Tower foundations	$5 456 000	
Anchorages	2 642 800	
Shore piers	753 600	
Substructure		$8 852 400
Metallic steel towers	$1 912 000	
Wire work, etc	4 993 661	
Suspended superstructure, etc	4 492 800	
Superstructure		11 398 461
Total		$20 250 861

For purposes of inspection an elevator ought to be placed in each of the four towers, and two of these elevators ought to be of sufficient size to accommodate passengers; $100 000 should be reserved for these elevators and the various appliances in connection with them.

The ornamental work on top of the towers, with provisions for lighting, etc., would cost another $100 000.

The structure, with a 10% allowance for contingencies and engineering, would cost about $22 500 000, or somewhat less than $5 500 a foot for the 4 100 ft. of suspended superstructure.

By making some modifications in the plan, among which may be mentioned allowing a greater flexibility under extreme conditions and

reducing the depth of the stiffening truss, the cost could probably be reduced to about $20 000 000.

TIME.

The time it would require to construct such a bridge would depend largely on the resources of the company building it. If everything were in readiness, both legally and financially, it ought to be built in five years. The foundations for the towers could be conducted simultaneously and completed in two years. The steel towers could be erected in another year. The anchorages and shore piers could be completed before the towers are done. The cables, being already manufactured, could be erected in one year. The back spans and projecting cantilevers could be raised while the cables were being put in position. The suspended portion of the superstructure, 2 800 ft., could be erected in one year. This allows two years for the erection of the metallic towers and the placing of the cables, which could probably be materially reduced. Five years would therefore appear to be enough for the construction of this bridge.

APPENDIX A.* TEST OF WIRE ROPES, MADE AT U. S. ARSENAL, WATERTOWN, MASS., MAY 2D-4TH, 1895.

In the straight wire ropes the elongation was measured on 100 ins.; in the coiled wire ropes, on 200 ins.

The numbers of the tests show the order in which they were made. The general practice followed was, first to apply an initial strain of 10 000 lbs. per square inch, the recording gauge being attached after this load was applied. The strains were then increased by successive increments of 10 000 lbs. up to 60 000 lbs., when the rope was allowed to rest for at least ten minutes while a number of observations were taken. The strains were then reversed back and forth six times between 50 000 and 60 000 lbs., raised to 70 000 lbs., reversed back and forth between 60 000 and 70 000, and then gradually raised by 10 000-lb. increments until the stretch made it necessary to remove the gauge; after each increment the strain was reduced 10 000 lbs. before increasing. In a few of the tests first made, this method was not fully carried out. In every instance except those designated by stars, the percentage of elongation recorded in the table was observed after the rope had been subjected to a strain 10 000 lbs. greater than the recorded strain.

Test No. 8 275 was left under a strain of 60 000 lbs. per square inch for 16½ hours, at the end of which time this strain was reduced by

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* A full report of these tests is on file in the Library of the Society.

TEST OF STEEL WIRE ROPES.

CHARACTER OF ROPE.	STRAIGHT WIRE.				COILED ROUND WIRE.				PATENT LOCKED WIRE.			
Character of wire	Special Steel.		Plow Steel.		Special Steel.		Plow Steel.		Special Steel.		Plow Steel	
Number of test...........	8 269	8 279	8 271	8 274	8 270	8 278	8 273	8 275	8 272	8 280	8 276	8 277
STRAIN PER SQUARE INCH.	PERCENTAGE OF ELONGATION ABOVE THAT PRODUCED BY INITIAL STRAIN OF 10 000 LBS. PER SQUARE INCH.											
50 000 lbs................	0.1525*	....	0.1834	0.1803	0.2588	0.2752	0 2257	0.2355	0.2735	0.2525	0.2742	0.2710
60 000 "	0.1994*	....	0.2255*	0.2195	0.3190	0.3353	0.2794	0.2817	0.3274	0.2973	0.3286	0.3259
70 000 "	0.2405*	....	0.2711	0.2604	0.3968	0.4059	0 3351	0.3345	0.3862	0.3553	0.3937	0.3877
80 000 "	0.2889*	....	0.3118	0.3032	0.4674	0.4811	0.3980	0.3937	0.4573	0.4229	0.4673	0.4611
90 000 "	0.3382*	....	0.3478*	0.3476	0.5509	0.5637	0 4676	0.4575	0.5424	0.5207	0.5537	0.5479
100 000 "	0.3869*	....	0.3956	0.3930	0.5930*	0.6724	0.5345	0.5209	0.5862*	0.6007	0.6496	0.6555
Ultimate strength........	150 000	146 640	188 980	187 360	148 120	140 000	187 230	177 690	133 180	131 970	142 220	176 850

about 330 lbs. per square inch, when the tests were resumed; the elongations given were all measured subsequently to this rest.

Test No. 8 280 was left under a strain of 60 000 lbs. per square inch for 39 hours, during which time the strain was reduced about 5 000 lbs. per square inch, when the tests were resumed; the elongations given were all measured subsequently to this rest.

Each of the round wire ropes was formed of 37 No. 8 wires. Each of the locked ropes was formed of 62 wires, of which the central one was round, the intermediate wires square and the outer layer of special lock section. The straight wire ropes were wrapped with fine soft wire.

In the cases of the two ropes which were left under strain, one over night and the other over two nights and an intermediate Sunday, the strains were reversed back and forth between 50 000 and 60 000 lbs. several times when testing was resumed, and the observations under these conditions were specially interesting and valuable. They were as follows:

Strain per Square Inch.	Elongation in 200 Ins.	
	Number of Test, 8 275.	Number of Test, 8 280.
60 000 lbs.	0.5510	0.5852
50 000 "	0.4711	0.5051
60 000 "	0.5511	0.5853
50 000 "	0.4711	0.5051
60 000 "	0.5513	0.5851
50 000 "	0.4711	0.5051
60 000 "	0.5512	0.5852

These show an extraordinarily uniform modulus of elasticity of 25 000 000 lbs., and show how uniformly this rope may be depended upon for action in a structure, even though there be a material difference in the quality of the wires. These observations were made prior to those recorded in the preceding table. The elongation for the 50 000-lb. strain is the same in the two tables. The difference between the elongations for the 60 000-lb. strains in the two tables is due to the set taken when the strain was raised to 70 000 lbs. before the 60 000-lb. strain was observed for the first table.

Five wires were taken from two of the straight wire cables, one being of special steel and the other of plow steel. The samples taken from the special steel showed an average strength of 172 588 lbs. per square inch, and an average reduction of 44.2 per cent.; the plow steel showed an average strength of 226 504 pounds per square inch and an average reduction of 45.7 per cent. In the case of the plow steel one wire was nicked before testing; its strength was fully up to the average of the others, but its reduction was so much less that it has been excluded in calculating the average reduction.

The fractures always occurred first in the outer wires and the ropes evidently failed to develop their full strength owing to defects in sockets, which were not as well finished inside as they should have been.

DISCUSSION.

T. C. Clarke, M. Am. Soc. C. E.—This paper is both interesting and instructive. It shows what to do, and what not to do. Mr. Clarke.

The system of socketed connections devised by the author has enabled him to plan the best system of anchorage known to the speaker. It has the great merit of enabling all the parts to be accessible for inspection and painting. On the other hand, the severing of the cables at the top of the towers seems unnecessary. It is true that it gives shorter and lighter parts to handle, but if a strand 3 342 ft. long can be handled, one 5 698 ft. long can also be handled. Its weight, taken at 10 lbs. per ft., is 28½ tons, which is not greater than that of street railway cables which have been transported by railway cars and wagons.

The adoption of a unit made of a number of wires instead of a unit of one wire is good, and will save time and interest during construction. Engineers may differ as to the plan of a twisted wire rope, but all must agree that this Society owes its thanks to the author for having made and reported his interesting experiments at the Watertown Arsenal.

The design of the towers is excellent in every respect. They give the greatest possible strength with the least material.

The speaker strongly objected to a stiff suspended girder, either with or without a hinge in the center. Such a design tries to unite two systems which are always working against each other, and the connections of the suspended girders, being the weakest parts, must give way sooner or later. The author's girder has reversal strains of 40 000 lbs. per inch in all, and he admits that ordinary steel will not do.

The only possible way to insure the durability of a suspended girder is to treat it merely as a distributor of concentrated loads, as Roebling did in his bridges. He purposely made his girders shallow and flexible. A flexible girder, however, is unsuited to the heavy trains and rapid speeds required by railway traffic. A better plan is to brace between the cables, as Gustav Lindenthal, M. Am. Soc. C. E., has done in his design. For such long and heavy spans, it is possible to utilize the cables themselves as chords without excess of material. Such a plan is more economical than the deep and rigid suspended girder. Engineers do not seem to realize how great the economy of this braced cable construction is. A comparison between that part of the designs of Mr. Lindenthal and the author will show this clearly.

Mr. Clarke.

COMPARISON OF DESIGNS FOR SUSPENSION BRIDGES, HUDSON RIVER AT NEW YORK.

DESCRIPTION.	Engineer Commission. August 23d, 1894.	G. S. Morison, Paper. October 4th, 1896	G. Lindenthal. July 20th, 1894.	W. Hildenbrand. July 20th, 1894.	Board of U. S. Engineer Officers. October 25th, 1894.
Length of land span	965	1 000	1 850	500	900
" of main span	3 220	3 200	3 100	3 310	3 200
" of land span	965	1 000	1 850	500	900
Total length between anchorages	5 150	5 200	6 800	4 310	5 000
Number tracks and load per foot	6 x 3 000 lbs.	8 x 1 500 lbs.	8 x 3 000 lbs.	6 x 3 000 lbs.	6 x 3 000 lbs.
Total load per foot	18 000 "	12 000 "	24 000 "	18 000 "	18 000 "
Strain per square inch on wire of 180 000 lbs., ultimate strength	60 000 "	58 921 "	65 000 "	50 000 "	60 000 "
Reversal strains suspended girder	12 500 x 2 lbs.	20 000 x 2 lbs.	?	15 000 x 2 lbs.	?
Tons wire work	42 104	30 160	57 800	42 230	28 159
Do. per lineal foot	8.17	5.80	8.50	9.80	5.63
Tons suspension girder.	50 565	52 160	37 260	28 110	26 156
		4 100			
Do. per lineal foot	9.80	10	5.5	6 7	5.23
Do. suspended	92 569	82 320	95 060	71 040	54 315
Do. per lineal foot	18	16	14	16.5	10.86
Total tons cables, girders, towers, anchorages, etc.	145 438	110 950	125 700	118 940	108 912
Tons per lineal foot	28.24	21.3	18.5	27.6	21.78
Cost, superstructure	$16 036 111	$11 270 949	$11 072 800	$12 079 700	$10 402 540
" substructure	17 480 960	8 852 400	9 927 200	3 830 000	11 784 000
" miscellaneous		2 375 756			
" total	$33,517 671	$22 500 00	$21 000 000	$15 900 700	$22 186 540
Interest, 5% for five years on half above amount	4 189 625	2 812 500	2 225 00	1 988 625	2 773 317
Total cost	$37 707 296	$25 312 500	$23 225 000	$17 890 325	$24 959 857
Cost per lineal foot between anchorages	7 321	4 870	3 415	4 151	4 990

Mr. Clarke.

DESIGN BY G. S. MORISON.

Cables..........	6.13 tons per foot at $140	$858	
Suspended girder.....	12.70 " " " " 86	1 092	
Total..................	18.83 " " "	1 950	
	$1 950 × 4 100 =		$7 995 000

DESIGN BY G. LINDENTHAL.

Cables and bracing ...	8.5 tons per foot at $140	$1 190	
Floor stiffener, girder.	5.5 " " " " 80	440	
Total..................	14.0 " " "	1 630	
	$1 630 × 4 100 =		6 683 000
	Difference...............		$1 312 000

When it is considered that Mr. Lindenthal's cables carry a moving load of 12 tons per foot, or double that of Mr. Morison's cables, which carry 6 tons only, the economy of the braced cable system becomes evident.

As it is an open secret that Mr. Morison's design was made for the crossing of the Hudson River at New York, a comparison between it and the designs which were submitted to and by the Engineer Commission and the Board of U. S. Engineer Officers in 1894 will be both interesting and instructive. The table on the preceding page is instructive as showing how experts can differ in working out the results of the same problem.

If the moving load taken by Mr. Lindenthal were reduced from 12 tons to 9 tons per lineal foot as assumed to be sufficient by the Engineer Commission, the saving of live load would be 19 800 tons. This would not only reduce the weight and cost of the cables, but also that of girders, towers, anchorages and foundations, and the whole saving would be at least 15 per cent. on the cost of bridge.

Mr. Lindenthal estimates the cost of his bridge,

if 5 500 ft. long, at..........................	$17 800 000
deduct 15%.........................	2 670 000
	$15 130 000
Add interest for five years at 5% on half the cost	1 891 000
	$17 021 000

This is less than half the sum estimated by the Commission of Engineers; but a careful examination of Mr. Lindenthal's quantities and prices fails to discover any serious error.

If this estimate is correct, it means that the cost of a six-track suspension bridge over the Hudson River at New York, between Sixtieth and Seventy-second Streets, would not exceed the probable cost of a cantilever bridge designed to carry 3 000 lbs. live load on each of six tracks, having a central span of 1 600 ft. and two side spans of 800 ft. each, supported by two piers in the river. Mr. Lindenthal deserves the credit of having been the first to demonstrate this by a plan which, if built, would be a work of real architectural merit.

Mr. Mayer. JOSEPH MAYER, M. Am. Soc. C. E.—The design for a suspension bridge made by the author differs in several important points from that of the Union Bridge Company for a suspension bridge of the same span accommodating six railroad tracks. The Union Bridge Company's bridge has twelve cables lying in vertical planes, and two suspended stiffening trusses in each half span, hinged at the center and ends. The stiffening trusses have curved top chords, and are 200 ft. high in the center; they do not project beyond the towers. In the construction of the cables, the Union Bridge Company followed American precedents. In the author's design the cables are cradled, the stiffening trusses are continuous at the towers and in the center, they have parallel chords, and are only 66 ft. 8 ins. high.

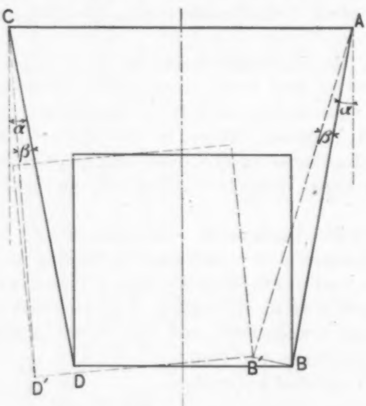


FIG. 20.

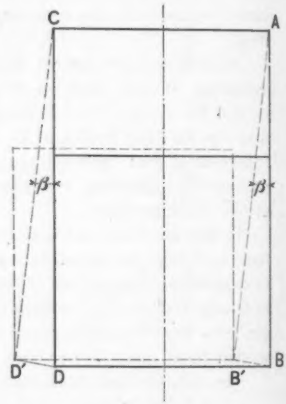


FIG. 21.

In the arguments brought forward in favor of the cradling of cables it is always shown how stable against lateral forces a suspension bridge is if it has cradled cables, and it is tacitly assumed that a suspension bridge, with the cables not cradled, is less stable in this respect. Fig. 20 shows the cross-section of a suspension bridge with cradled cables; the lines $A\,B$ and $C\,D$ give the planes of the cables, which form angles α with the vertical plane through the axis of the bridge. If the wind pushes the bridge aside until the cables are in the planes $A\,B'$, $C D'$, then the horizontal force which it opposes to the wind is

$$H = \frac{P}{2} \tan(\alpha + \beta) - \frac{P}{2} \tan(\alpha - \beta),$$

where P is the suspended weight of the bridge and β the angle of the planes of the cables in the new and old positions.

If the bridge is not cradled and is moved sideways by the wind until the planes of the cables form an angle β with their original position, then the horizontal force which it opposes to the wind is $H' = P \times \tan \beta$ (see Fig. 21). Mr. Mayer.

To find what these values are in concrete cases, take the cradling to be 42.5 ft. (equal to the average cradling in the author's design); the versed sine of the cables, 400 ft.; the arc $B\,B' = D\,D' = 8$ ft.; tan α is then $\frac{42.5}{400} = 0.10625$ and $\alpha = 6^\circ\ 3'\ 54''$.

$$\beta^\circ = \frac{8 \times 180}{402.25 \times 3.14159} = 1^\circ\ 8'\ 22''$$
$$\tan \beta = 0.01989$$
$$\alpha + \beta = 7^\circ\ 12'\ 16''$$
$$\alpha - \beta = 4^\circ\ 55'\ 32''$$
$$\tan \alpha + \beta = 0.12641$$
$$\tan \alpha - \beta = \underline{0.08617}$$
$$\tan (\alpha + \beta) - \tan (\alpha - \beta) = 0.04024$$
$$\underline{2 \tan \beta = 0.03978}$$

The horizontal force opposed to the wind after a lateral deflection of 8 ft. has taken place, is—

In the bridge with cradled cables $\frac{P}{2}$ 0.04024

" " parallel " $\frac{P}{2}$ 0.03978

The difference between these two amounts is $\frac{P}{2}$ 0.00046, or about 1.14% of the total amount.

It is claimed that this argument does not indicate the value of cradling for small deflections, but that it does is shown by the following computations for the two types of bridges. Taking the same general dimensions as before and assuming that the lateral deflection is only 1 ft., β will be $8'\ 33''$ and α $6^\circ\ 3'\ 54''$. $\alpha + \beta = 6^\circ\ 12'\ 27''$ and $\alpha - \beta = 5^\circ\ 55'\ 21''$. Tan $(\alpha + \beta) = 0.108766$ and tan $(\alpha - \beta) = 0.103737$. Tan $(\alpha + \beta) -$ tan $(\alpha - \beta) = 0.005029$; 2 tan $\beta = 0.004974$; tan $(\alpha + \beta) -$ tan $(\alpha - \beta) - 2$ tan $\beta = 0.000055$. In this case the difference in resistance to wind of the two bridges is only 1.09% of the total amount, even less than it is for the larger deflection of 8 ft.

It has been stated that at the beginning of the deflection, the center of gravity of the bridge with cradled cables would rise more than that of a bridge having cables in vertical planes. The importance to be attached to this argument can be shown in the following manner: Take the general dimensions as before and let β be the angle of motion. In the bridge with cradled cables, one side rises by an amount equal to 402.25 sin $\left(\alpha + \frac{\beta}{2}\right) 2 \sin \frac{\beta}{2}$, and the other side falls by

Mr. Mayer. an amount equal to $402.25 \sin\left(\alpha - \frac{\beta}{2}\right)\ 2 \sin \frac{\beta}{2}$. The rise of the center of gravity of the bridge, assuming the loads on both cables are alike, is $402.25\left(\sin\left(\alpha + \frac{\beta}{2}\right) - \sin\left(\alpha - \frac{\beta}{2}\right)\right) \sin \frac{\beta}{2}$. If the cables are not cradled, the rise of the center of gravity is $400 \times 2 \sin^2 \frac{\beta}{2}$.

Since for small angles the sines can be taken equal to the arcs, the rise in the two cases is approximately $402.25 \left(\frac{\beta}{2}\right)^2 \times 2$ and $400 \left(\frac{\beta}{2}\right)^2 \times 2$, which shows that the difference in this respect between the two types of bridges is insignificant.

Another objection raised against vertical cables is that the resistance to wind from the inertia of the mass of the bridge will be less than when the cradling is employed. The inertia of a mass shows itself in the resistance it opposes to any change in velocity, and the force necessary to overcome the inertia is equal to the mass multiplied by the acceleration. Since there is slightly more motion, for equal lateral deflection of floor, in the upper parts of the stiffening trusses if the cables are cradled than if they are parallel, the influence of the inertia is more in the first case than in the second; but this influence acts against the wind at the beginning of the motion when the wind strains are small, and with the wind toward the end of the motion when the acceleration is negative and the wind strains are largest. The effect of inertia is therefore to increase the wind strains, and not to decrease them. The effect of inertia is, however, small while the bridge is in motion, and is nil as soon as the bridge has assumed a position of equilibrium with the wind pressure.

The effect of inertia on bridge strains, except that produced by vibrations, is apt to be exaggerated. To show this, assume six trains 1 400 ft. long to move at 60 miles an hour, or 88 ft. a second, across the author's bridge. From the moment when they cover the left half of the span to the moment when they cover the right half, a point 700 ft. to the right of the center will sink about 7 ft. For this motion it has $\frac{1400}{88} = 16$ seconds; since the velocity of this motion at the beginning and end is 0, the average velocity, $\frac{7}{16} = 0.4375$ ft., will be about half the largest velocity; this latter will therefore be 0.875 ft. This largest velocity will exist about in the middle of the motion; it is therefore acquired in 8 seconds. The average acceleration during the first part of this motion will therefore be $\frac{0.875}{8} = 0.1094$ ft. The largest acceleration will be about twice this amount, or 0.22 ft., and

the resistance which a weight P offers to this acceleration is equal to $\frac{P}{32.2} 0.22 = 0.007\ P$. This is less than 1% of the weight of the mass considered, and can, therefore, very properly be neglected. Mr. Mayer.

Unless it is asserted that the wind will produce much larger accelerations in the mass of the bridge than would be produced by these trains, it follows that the effects of inertia can be entirely neglected in the consideration of the wind strains.

This shows that a suspension bridge offers practically the same amount of lateral resistance to wind whether the cables are cradled or not. This lateral resistance does not arise from the cradling, but from the suspension. The design becomes simpler in many respects if the cables are in vertical planes instead of cradled, and offers practically the same amount of lateral resistance to wind. The Union Bridge Company, therefore, chose the former arrangement.

In the preceding argument two cables have been assumed, but it remains valid for any number of cables, provided those on the same side of the bridge are in parallel planes. With this arrangement the strains in the cables from wind are practically nil.

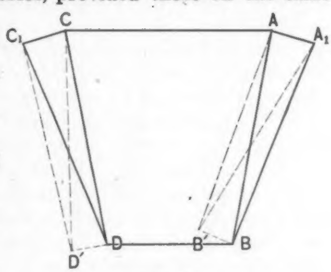


FIG. 22.

With the arrangement the author has chosen, the cables become unequally loaded when there is a lateral deflection of the bridge. If the cables are of the same length before the deflection, the outside cable to the windward and the inside cable to the leeward will be longer after the deflection than the other two (see Fig. 22). In consequence the load on the former cables will be increased, and that on the latter will be decreased the same amount. The former cables will lengthen and the latter will shorten an equal amount. The lines DD' and BB' will be arcs with centers midway between CC_1 and AA_1.

The versed sine of the cables will change by an amount approximately equal to $\frac{8 \times 14}{400} = 0.28$ ft., taking the dimensions of the author's bridge and a lateral deflection of 8 ft. in the center of bridge. This same deflection of the cables would be produced by a uniform load of 770 lbs. per lineal foot of bridge.

The horizontal force per lineal foot of bridge opposed to the wind in consequence of this unequal loading of the cables is $\frac{770}{4} \times \frac{28}{400} \times 2 = 27$ lbs. The approximate result is therefore that an unequal loading of the cables is produced in consequence of the fact that the two cables on each side of the bridge are not in parallel planes. This un-

Mr. Mayer. equal loading of the cables produces the same strain as a uniform vertical load of 770 lbs. per lineal foot of bridge and balances a wind pressure of only 27 lbs. per lineal foot of bridge.

The unequal cradling of the two cables on each side of the bridge produces, therefore, an increased stability, since the lateral force opposed to the wind is increased by 27 lbs. per lineal foot, but this increased stability is obtained at the cost of strains in the cables which are 28½ times as large as those which would be produced by a vertical load of 27 lbs. per lineal foot of bridge. The unequal cradling of cables is therefore an uneconomical way of resisting wind pressure and is a worse arrangement than the equal cradling of all cables.

In this respect the design in the paper is, however, a great improvement over the design submitted in the report of the Board of Engineers appointed by President Cleveland to determine the longest safe and practicable span for a Hudson River bridge at New York. That design proposed twelve cables, six on each side of the bridge, 20 ft. apart on top of the tower (see Fig. 23).

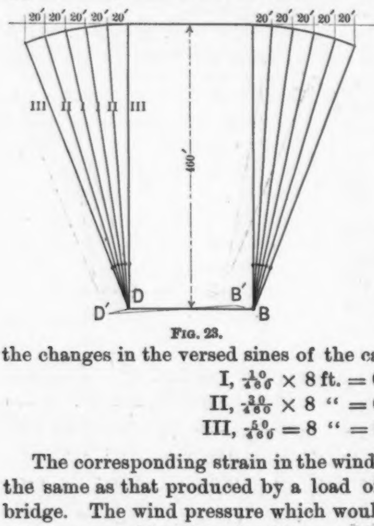


FIG. 23.

The lateral deflection $B\,B'$ and $D\,D'$ would take place on a line approximately perpendicular to the center line between the planes of the six cables. If the lateral deflection were equal to 8 ft., then the changes in the versed sines of the cables would be approximately

$$\text{I, } \tfrac{10}{460} \times 8 \text{ ft.} = 0.174 \text{ ft.}$$
$$\text{II, } \tfrac{30}{460} \times 8 \text{ “} = 0.522 \text{ “}$$
$$\text{III, } \tfrac{50}{460} = 8 \text{ “} = 0.87 \text{ “}$$

The corresponding strain in the windward cable III would be about the same as that produced by a load of 2 400 lbs. per lineal foot of bridge. The wind pressure which would be balanced in consequence of this unequal loading of cables would be 61 lbs. per lineal foot of bridge. That is, 1 lb. of wind pressure balanced on account of the unequal cradling of cables would produce the same strains on some of the cables as about 40 lbs. of vertical load.

The next point in which the bridges of the author and the Union Bridge Company fundamentally differ is the design of the stiffening trusses.

The argument by which the author tries to show that it is advantageous to use a shallow stiffening truss is substantially correct in so far as it asserts that the shallower the stiffening truss, the larger a

percentage of the unequally distributed load is carried by the cables without producing strains in the stiffening truss, and, therefore, the smaller the load to be distributed by the latter. It is defective in so far as it asserts that a shallow stiffening truss carrying a certain load for a given bridge is necessarily lighter than a high stiffening truss carrying a greater load for the same bridge. All depends in this case on the amount of difference in the two loads and the amount of difference in the depth of the two trusses. **Mr. Mayer.**

There are cases where the uniform load is large in comparison with the unequally distributed load, and where the versed sine of the cables is small. in which the deformation of the cables produced by the unequally distributed load is also small; in these cases a small decrease in the depth of the stiffening truss produces a large decrease in the load it has to distribute, and it is then economical to make the stiffening truss as shallow as the allowable deformation permits.

There are other cases, especially railroad bridges, where the unequally distributed load is comparatively large, and where the use of steel towers permits a large versed sine of cables; in these the deformation of the cables produced by the unequal loading is large, the allowable deformation of the bridge produces a small diminution in the load of the stiffening truss, and necessitates a large deviation from the most economical depth; and in these a shallow stiffening truss becomes uneconomical because the loss on account of small depth is more than the gain on account of small load. The case discussed in the paper is of this class. A stiffening truss 200 ft. deep in the center, carrying nearly the whole unequally distributed load, is lighter than a stiffening truss 66 ft. 8 ins. deep, carrying 69.4% of the unequally distributed load. The stiffening truss of the author's bridge, if it were 200 feet deep throughout, would have to distribute 90.5% of the whole unequal loading. If it were lower toward the ends, and if it had a hinge at each end and in the center, it would deflect considerably more than if it were continuous and had parallel chords. Ninety per cent. is a safe estimate for the unequally distributed load which has to be carried by a stiffening truss of the shape the Union Bridge Company has adopted.

It is therefore necessary to compare a stiffening truss 200 ft. deep at the center of the half span, hinged at the center of the span and at the ends, with curved top chords, carrying 90% of the unequally distributed load, with a stiffening truss 66 ft. 8 ins. deep, continuous at the center and ends, carrying 69.4% of the unequally distributed load. The former truss has a load 30% larger than the latter.

The estimate of weight of the latter truss gives—

11 100 lbs.	per	lineal	foot	for the	chords.
3 240	"	"	"	"	webs.
690	"	"	"	"	cross-braces.
750	"	"	"	"	top laterals.
15 780 lbs. total.					

Mr. Mayer. The section required in the truss 200 ft. deep would be one-third of the section required for truss 66 ft. 8 ins. deep, if the load were the same. The load being 30% larger, the section wanted is 43% of the section of the truss 66 ft. 8 ins. The web strains in the high truss would be 30% more than in the low one if the chords were parallel; the top chords being curved, the web strains are largely reduced at the ends.

A fair estimate gives for the high truss a saving of 54% in the weight of the chords, an addition of 60% in the weight of the webs, and an addition of 100% in the weight of the cross-bracing, or

Decrease in chords........	5 994 lbs.	per lineal foot of bridge.
Increase of web...........	1 944 "	" "
Increase in cross-bracing..	690 "	" "

This is a net saving of 3 360 lbs. per lineal foot of bridge in the stiffening trusses, or about 21.3% of the weight of the stiffening trusses.

If a hinge is made in the center there are practically no strains from changes of temperature or from deflection of cables under load. If this hinge is made so that it allows lateral motion as well as vertical motion, then the lateral motion will be so large, without being more than permissible on so long a span, that the cables take the largest part of the uniformly distributed wind pressure. How important these two advantages of a center hinge are can be seen from the fact that the strains produced in the center of the chords of the stiffening trusses by the effects of temperature changes and load changes in the length of the cables, amount to about ± 5 000 lbs. per square inch, and the strains from wind amount, according to the author, to ± 14 000 lbs. per square inch of bottom chords. These strains, together, are therefore larger than all the other strains at the centers of the chords of the stiffening trusses.

An objection has been raised to the use of a hinge in the center of the span, permitting lateral as well as vertical motion, that it would be difficult to make such a detail. If the lateral deflection is so large that the cables balance the wind pressure near the center of the span, then this hinge will have to resist no tension or compression but only shear, and there are several methods of constructing it. One of them is connecting the ends of the top chords of the first half span with the ends of the bottom chords of the second half span, and the ends of the top chords of the second half span with the ends of the bottom chords of the first half span, by means of vertical wire ropes which will transfer shear in both directions, and will allow considerable lateral deflection, especially if they have considerable length, without producing large tensions or compressions in the chords of the stiffening trusses. The same connection might be made by means of flexible plates instead of wire ropes.

In the preceding comparison of the weight of two stiffening trusses, one 66 ft 8 ins. high and continuous, the other 200 ft. high and hinged, no consideration has been taken of the very large strains in the chords of stiffening trusses, which are the consequence of the absence of hinges, and would, if considered, make the comparison still much more favorable to the hinged truss. The other differences between the strains of a hinged truss and those of a continuous truss have also been neglected through the whole argument. These differences are not very large, and are, on the whole, in favor of the hinged truss. Mr. Mayer.

In the argument showing that the stiffening truss has to distribute only part of the moving load, in the author's bridge, 69.4% of it, it is assumed that the other 30.6% of the moving load will produce at the lower ends of the suspenders exactly the same deflections as the 69.4% of it produce at the corresponding points of the stiffening trusses. This assumption is based on the single fact that the deflections produced at one-quarter of the span by the two loads before mentioned at the ends of the suspenders and in the stiffening truss are alike. Since the deflections produced in the cables and stiffening trusses by any loads are dependent on different and independent variables, this assumption cannot be absolutely and generally true. This vitiates to a large degree the calculation of a shallow stiffening truss and to a much smaller degree that of a deep one. A considerable saving, which has not been considered, can be made by varying the sections throughout the length of the stiffening trusses, but this is only advisable where it is possible to make accurate strain sheets, as is the case in a hinged truss.

The foregoing reasons induced the Union Bridge Company to choose, in its design for a suspension bridge across the Hudson for six railroad tracks, cables in vertical planes and hinged stiffening trusses of 200 ft. depth.

The difficulty in the author's design for the cables, which arises from the impossibility of making sockets as strong as the ropes, might be overcome by bending the latter about a half-round saddle on top of the towers before inserting them into the sockets. In this case the maximum strains in the ropes at the sockets would probably be so much smaller than near the saddles that the sockets would not be the weakest parts of the cables. Certainty on this matter could be obtained easily by tests.

F. Collingwood, M. Am. Soc. C. E.—The author states that the strands of the cables will be adjusted under tension at the works, so that no further adjustment will be required in the field. The work of putting the ropes in place is described, and apparently this is to consist of simply carrying them across the span one by one, and placing the sockets in the places provided for them. In this connection the speaker asked whether consideration has been given to the change in Mr. Collingwood.

Mr. Collingwood. elevation which will be caused by the change in length, resulting from drawing in the ropes to form the cables. The diameter at the points of attachment scales in the cuts in the neighborhood of 7¼ ft., and allowance must be made in the length of each strand for the position it finally occupies.

Again, it is problematical whether it is practicable to bring numerous ropes hanging freely in space into a compact cylindrical form without building up a core first, and possibly adding successive layers. The lateral movement of any one of the many ropes will interfere with the final result, and such movement is almost certain to be caused by the operation of compression. The fact, also, of the wires in the ropes twisting spirally around the exterior will interfere with the free movement which straight wires would have.

In laying up the cables of the existing East River Bridge, the strands were each seized at intervals of 2 ft. 4 ins. throughout their lengths. After the first twelve strands in each cable were completed and in place, the central seven were made into a core. In each case the previous seizings were removed sufficiently far in advance to allow the wires to be spread out into their true position, which was done by means of mallets and pressure. Finally the nineteen strands were brought into a compact whole.

The exact length of every strand when in position in the cable was calculated as nearly as possible, care being taken not to make it too short. When the strand had been lowered to place, its length was finally adjusted by the position it occupied vertically at the center of the span, those strands which were to be drawn in the most hanging the lowest, the central strand being the guide.

The work of adjustment is undoubtedly increased by cutting the ropes at the towers and the increased weight of material due to it is a large item; the question naturally arises as to whether it is a necessity.

Mr. Cooper. THEODORE COOPER, M. Am. Soc. C. E.—In discussing this thoughtful paper, it should be borne in mind that the number of tracks, character of the train service and local conditions will vary for different localities, and that even at any particular locality there may be a wide difference in the views held as to the capacity that should be given to such a structure.

The only true test of the relative merits of different systems of bridges to accomplish best the desired result is a comparison of costs for structures under a uniform set of requirements.

The author has adopted a cable system formed of coiled-wire ropes, a stiffening truss continuous over the piers, and a maximum allowed deflection at the quarter-span, as essential features of his study. Whether this combination will give an economical and satisfactory structure is at least questionable.

Coiled-wire ropes have been used for suspension bridges, and have attractive features from the point of view of more rapid erection of the cables. The longest span formed of coiled-wire ropes known to the speaker is the Middle Harbor Bridge, at North Sydney. It is 1 026 ft. long, and has, judging from a photograph, a center-hinged stiffening truss; it is not a railroad bridge. Cables of this kind have been advocated by General E. W. Serrell for bridges over the Hudson River at Fort Montgomery and at New York City. Mr. Cooper.

Coiled-wire ropes must have a lower modulus of elasticity than cables formed of straight wires. While the elongation which measures the modulus is in the straight wire an actual elongation of the metal, in a coiled rope it is only partially an elongation of the metal, the rest being due to the elongation of the coils of the individual wires forming the rope. The elongation of the coils will depend, to a more or less extent, upon the rigidity of the core and degree of hardness with which the coils are wound, for the only thing which prevents the straightening out of the coils is the radial resistance to compression of the core and inner strands.

The modulus of a coiled-wire rope is partly tensile elongation of the metal, partly torsion, and partly elongation of the coils.

It is natural, therefore, to expect coiled ropes to have a considerably lower modulus than straight wires, and also that the moduli would be variable for different ropes or parts of the same rope.

As far as the speaker has been able to discover, actual tests confirm the above theoretic expectations. Tests submitted to him by General Serrel, with his proposed plan for the North River Bridge, which were made at the Watertown Arsenal, showed that the moduli of different ropes for different loads varied as widely as 12 500 000 to 24 000 000 lbs. Between the loads of 30 000 to 60 000 lbs. per square inch a hawser-laid rope gave a modulus of 16 500 000 lbs., and ropes formed somewhat similar to those proposed by the author gave moduli from 21 500 000 lbs. to 24 000 000 lbs.

The abstract of tests given by the author shows that under different degrees of loading the elongations of the coiled-wire ropes exceeded by 30% to 60% the elongations of the straight ropes. The coiled ropes, however, after being under strain for some hours, gave a fair modulus of about 25 000 000 lbs.

The fuller reports* are very interesting, and make a better showing between the coiled and straight-wire ropes than the abstracts did. Test No. 8 271 on straight-wire rope of plow steel, and test No. 8 275 on coiled-wire rope of plow steel, may be taken as typical of the general results. The elongations for each 10 000 lbs. in percentages are tabulated on the next two pages.

* Sent by the author to the writer after the meeting. A copy of the complete report of the tests is filed in the Library of the Society.

Mr. Cooper.

No. 8 271.—Straight-Wire Rope of Plow Steel.

Strain per square inch. Pounds.	Elongation for 10 000 lbs. Percentage.	
10 000	0.000	
20 000	0.0632	
30 000	0.0432	
40 000	0.0386	
50 000	0.0410	
20 000	0.0517	
30 000	0.0473	
40 000	0.0428	
50 000	0.0416	
60 000	0.0406	
20 000	0.0416	
70 000	0.0447	
20 000	0.0483	
80 000	0.0428	
70 000	0.0359	
60 000	0.0364	
70 000	0.0361	Average 0.0362
80 000	0.0364	
90 000	0.0398	
80 000	0.0352	
70 000	0.0363	Average 0.0361
60 000	0.0368	
20 000	0.0580	
60 000	0.0447	
70 000	0.0372	
80 000	0.0364	Average 0.0370
90 000	0.0374	
etc.		

Ultimate strength of rope......... 188 980 lbs.
" " " wires .... 226 300 "

The negative set shown on relaxing the strain and the difference in ultimate strength between the rope and the wires from which it was formed indicate that the wires of the rope were not pulling equally. The elongations, therefore, are greater, and the resulting modulus less, than would be the case were all the wires equally strained. For long ropes this failure to pull equally would be less than for short samples suitable for use in the testing machine. The small differences in lengths of the wires and also any slipping of the wires in the sockets would be relatively less important, as the ropes increased in length.

It will be seen that when this rope had been strained to 80 000 lbs. and then tested back and forth from 90 000 to 60 000 lbs., the elonga-

tion became quite uniform at about 0.0361, giving a modulus of about 27 700 000 lbs. Perhaps this modulus would have been higher if this rope had been treated similarly to the coiled ropes 8 273 and 8 275, viz., kept under a strain of 60 000 lbs. for a number of hours and then tested back and forth between limits not exceeding 10 000 lbs. It would be very interesting to have had exactly similar tests upon both kinds of ropes and single wires, that their moduli could be compared. Mr. Cooper.

No. 8 275.—Coiled-Wire Rope of Plow Steel.

Strain per square inch. Pounds.	Elongation for 10 000 lbs. Percentage.	
10 000	0.000	
20 000	0.0520*	
30 000	0.0513*	
40 000	0.0527*	
50 000	0.0510*	
60 000	0.0527*	
50 000	0.0415	
60 000	0.0420	
50 000	0.0415	
60 000	0.0415	
50 000	0.0412	Average.........0.0416
60 000	0.0417	
50 000	0.0416	
60 000	0.0416	
50 000	0.0400	
60 000	0.0400	
50 000	0.0400	
60 000	0.0401	Average.........0.0400
50 000	0.0401	
60 000	0.0400	
70 000	0.0463*	
60 000	0.0403	Average.........0.0404
70 000	0.0408	
80 000	0.0517*	
70 000	0.0396	Average.........0.0402
80 000	0.0408	
90 000	0.0581*	
80 000	0.0397	Average.........0.0406
90 000	0.0415	
etc.	etc.	

Ultimate strength of rope.............. 177 690
" " wires............. 226 300

* Held at 60 000 lbs. for 16½ hours.

Mr. Cooper. It will be noticed that when a new strain greater than a previous strain is put upon the rope (as marked by*), the elongation is large; but when the strains are not varied more than 10 000 lbs. back and forth, the elongations are smaller and more uniform. The wires probably move slightly as long as the strains are increasing, making the greater elongations; but after they have taken on the new position, motion does not take place for limited strains of a less intensity. If the rope is held for hours under one strain, the wires complete their movement and then the elongation becomes the true elongation—free from any movement of the wires on themselves.

The wires in the coiled rope appear to have pulled together—as would be expected from their twisted condition—better than in the straight rope.

The modulus of 25 000 000 lbs. as compared to the modulus 27 700 000 lbs. of the straight rope is a favorable result, but in longer ropes or bridge cables it is probable that the coiled ropes would not show much better results, while the straight ropes should be somewhat better. The ultimates would probably be better in an actual cable, but the straight ropes would get the greater increase. Neither, however, could be expected to give as good an ultimate strength or modulus as would be due to straight wires. Judging from the ultimates of the straight ropes and the single wires, the modulus of the wires should have been 30 000 000 lbs. or more.

The author has selected a stiffening truss continuous over four supports, which will be about 25% stiffer than a truss supported at the ends. As the cables will be more elastic than straight-wire cables, the combination of a stiffer form of truss with the more elastic form of cable will not be in the line of economy, for it will increase the proportion of the load which must be carried by the truss, and also increase the strains due to the rise and fall of the cables through temperature changes. Whether the loss of economy will be compensated for by the absence of a central hinge and end fastenings is questionable.

The author's description of the functions of the stiffening truss for different classes of bridges does not cover the whole case. The speed of the moving load is also an important factor. A cable that might be sufficiently flexible to accommodate itself to slowly moving loads without injury would be seriously affected by the same loads moving at far higher speeds.

The author has selected as a permissible deflection at the quarter span a depression of 3½ ft., with its accompanying rise at the opposite quarter, for the purpose of proportioning his stiffening truss. That such a distortion of the cables would not in itself be harmful is true; but when such distortions, if they occur, must be reversed at opposite quarter spans in the space of a few seconds, under trains moving at 30

to 40 miles per hour, they become a serious matter. On the only railroad suspension bridge in use for heavy trains, the Niagara Suspension Bridge, trains are not only limited in weight, but also to very slow speeds, 4 to 5 miles per hour. Considering the limitation that this restriction puts upon the capacity of this bridge, it is fair to assume that they have not found high speeds favorable. **Mr. Cooper.**

It is not of course claimed that there will be undulations traveling from one end of the cables to the other of any such magnitude as $3\frac{1}{2}$ ft., as the inertia of the mass would not permit it; but the energy corresponding to such an undulation must be destroyed somehow. That this will not produce dangerous and destructive results in the cables or other parts of the structure should be well established before being adopted as a feature of the design.

That it would tax heavily the suspender system seems certain. For a railroad suspension bridge, reliance should not be placed on the frictional resistance of cable clamps for suspender loads. While it may be possible to obtain sufficient friction to resist the slipping of the clamps, the compression put upon the cables to obtain such friction may be very injurious to the cables. Moreover, any slipping of the clamps injures the cable wrapping. It is stated that some of the clamps of the Brooklyn Bridge slipped in the early days of its construction.

For long spans like the one under consideration, the greater versed sine, obtained by use of metal towers, makes the angle of the cables much steeper than those of the Brooklyn Bridge (about 50 per cent.).

In addition, the heavier train loads make it more important that the connection of the suspenders to the cables be positive.

In the specifications for the proposed North River Bridge, the speaker required that "the pull of the suspenders in the direction of the cables must be taken up otherwise than by the friction of the cable bands produced by bolts through their flanges. Only 20% of the normal pressure upon the cables, produced by the load on the suspenders, shall be considered as resisting this pull. The remainder must be taken up by supplementary cables or ropes running from the saddles to the suspender connections or some equally acceptable device."

Fig. 24 shows the general method that was intended by this clause. The inner sleeve is clamped to the cable as a bearing for the free saddle carrying the suspenders. The saddle is held by the supplementary ropes extending from one saddle to the other, and anchored at the towers. By this means the saddle can accommodate itself to any sudden or undue loading without the chance of causing slipping of the cable clamp out of position.

If the vertical load on one suspender is called V, the pressure in

Mr. Cooper. a normal direction at the suspender joint at the highest point (for the plan under discussion) will be 0.903 *V*, and the tangential pull 0.425 *V*. If 20% of the normal pressure is assumed to resist the tangential pull, the suspender load on the highest suspender only needs a tangential resistance of 0.244 *V*. Summing up all the tangential pulls for the whole series of suspenders, it will be found that, when all the suspenders are fully loaded, the maximum tangential pull at the towers will be about 3 *V*, or the supplementary ropes at the towers need be only sufficient to resist the load on three suspenders, or 100 ft. of

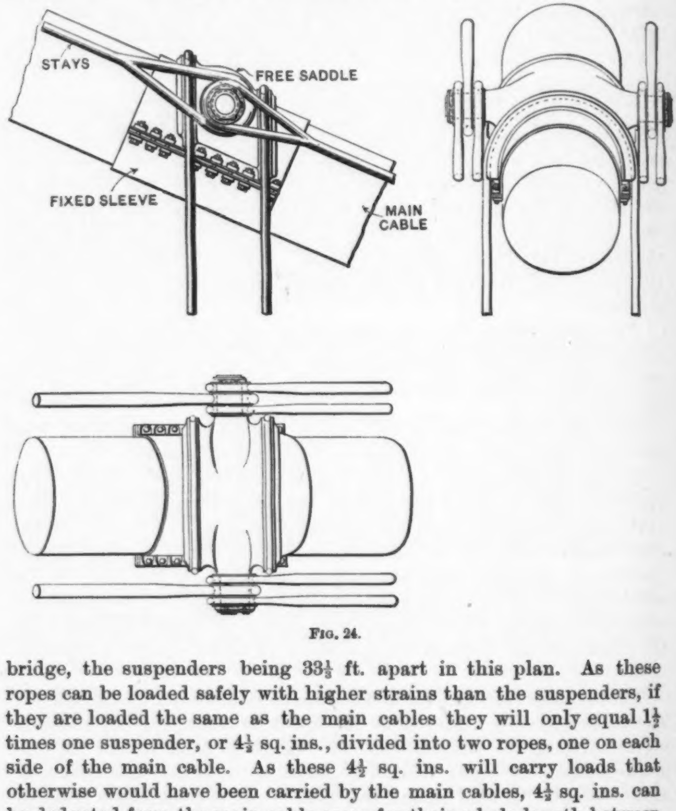


Fig. 24.

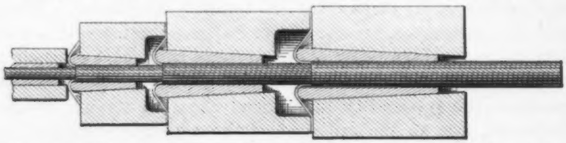
bridge, the suspenders being $33\frac{1}{3}$ ft. apart in this plan. As these ropes can be loaded safely with higher strains than the suspenders, if they are loaded the same as the main cables they will only equal $1\frac{1}{2}$ times one suspender, or $4\frac{1}{2}$ sq. ins., divided into two ropes, one on each side of the main cable. As these $4\frac{1}{2}$ sq. ins. will carry loads that otherwise would have been carried by the main cables, $4\frac{1}{2}$ sq. ins. can be deducted from the main cable areas for their whole length between towers, so that, instead of being an additional cost, they will produce a saving.

E. GYBBON SPILSBURY, M. Am. Soc. C. E.—Mr. Cooper calls attention to the fact that the results of the experiments on wire ropes mentioned in the paper do not correspond with those found by Gen. Serrell some years ago. The discrepancies between the results appear to be due to the difference of the structure of the ropes or cables tested, and also to the difference of the methods used in socketing. Gen. Serrell made his tests on a rope which he describes as hawser-laid. This rope, as generally known to the trade, is made up of six strands, each consisting of twelve wires twisted round a hemp center, these strands being again laid together round a hemp center. With a cable made in this manner, it is hard to see how any one could expect to obtain any regular or determined modulus. Even if the term "hawser-laid," used by Gen. Serrell, was a mistake, and he meant an ordinary six-strand cable with wire instead of hemp centers, the results would have been the same, as the tensions of the wires in each strand would be different from the tensions of the strands in the rope, and it would be a practical impossibility to bring them into unison. The twisted cables tested by the author were in reality simple strands, and consequently not subject to the same objections. In one set of these test ropes the wires were all twisted with a long lay in the same direction; the other set had each layer of wires coiled in opposite directions. The advantage of this latter construction is to balance the internal strains and obviate the natural tendency of coiled ropes to revolve on their central axis when under strain. Whether this advantage will be found to overbalance the possibility of internal shearing strains being set up in the cable, due to the crossing of the wires, as suggested by Mr. Cooper, the speaker was not prepared to say, nor did it seem material to the general question, as the results for modulus were practically the same in both cases. Mr. Spilsbury.

Admitting the feasibility of using coiled strands in such a bridge construction, the success of the socketing of these strands is of the highest importance. The author calls attention to the fact that this is possibly the weakest point of the structure, and bases this opinion on the result of the tests before alluded to. As a fact, the first breaking of the cables under test always occurred on the outside layer of wires, but inside the socket. It was evident from this fact either that the inside shape of the socket was defective, as suggested by the author, or that a certain amount of slippage took place in the inner layers of the rope, which, not being participated in by the wires of the outside layer, resulted in imposing an excess of strain on these latter, and their consequent breakage.

The sockets used were of the ordinary type for this kind of rope. They were cylinders of soft machinery steel bored out conically for the spread of the wires of the strand. The method of socketing was as follows. The end of the rope being passed through the socket, the outside

Mr. Spilsbury. layer of wires was partially untwisted, and each wire separated and spread out against the taper surface of the socket. A split wedge-shaped thimble, having the same taper as the socket, is then put around the inner coil and driven home. In the interstices between each wire are driven taper wedges. The first layer secured, the wires of the succeeding layers are similarly spread out, and all the interstices filled in with similar wedges and thimbles, all being finally forced down into place, either with a screw or hydraulic press. Theoretically, if the result of this work was perfect, the central wires, being the shortest, should have broken first; whereas, as stated by the author, the outside wires were the ones which failed. The author attributes the failure at this point to be due to the ridge or shoulder which existed in each coupling at the point where the taper merged into the straight opening. The speaker's impression is that this ridge cannot account alone, if at all, for the breakage, owing to the fact that the material composing the socket was soft steel, while the wires of the rope were excessively hard. This was shown by cutting open one of the sockets after the test, when it was found that all of the outside wires were imbedded to about one-third of their diameter in the steel of the socket, which, under the pressure, had flowed up between the wires, and in some



instances been forced up even between the wedges in a thin film. It is his opinion that the failure of the sockets used to develop the full tensile strength of the rope was due to the difference of the coefficient of friction between the wire and the metal of the socket, in the first place, and the wires among themselves, in the second place, thus allowing these latter to slip on themselves and impose excessive strain on the outside wires, which, under the increased frictional pressure, were unable to give correspondingly. In order to avoid these defects, a new socket has been designed, specially adapted for smooth-coil cables. This socket (Fig. 25), which has been named, from its shape, the steeple socket, is really a combination of several sockets one back of the other, and each one having to take care of only a certain number of layers of wire. The cable being passed through the first section of the coupling, the outside layer of wires is spread out to conform with the taper in the socket, while the rest of the rope remains intact and goes through to the second socket. Taper wedges and thimbles are driven in between these outer wires and the central core, and smaller wedges are driven down between each of the outer wires, thus insuring an even pressure on each of them. The ends of these wires being then bent

over the outside of the socket, the second socket is fitted down on them, and the second and third layers of rope are spread out in the taper of this socket, and secured in the same manner as the outside wires were in the first one. This is repeated in each following socket, until every wire in the rope has been individually secured. While the length of such a fastening is somewhat greater than the ordinary socket, it is not so much so as would be imagined, as the size of each following socket is, of course, much smaller than the preceding one, it having less work to perform. Several of these fittings have been tested on different sizes of ropes; the rope has broken invariably entirely outside of the sockets, thus showing that the fastenings were always stronger than the cable. **Mr. Spilsbury.**

W. H. Breithaupt, M. Am. Soc. C. E.—The cutting of the cables at the towers in the author's design involves a great deal of work and additional weight. To offset this it is held that a stiffer and stronger wire can be used in the cables on account of avoiding its weakening by the abrupt bend at the towers, that erection will be much simplified, and that the backstays can better be given any desired direction. **Mr. Breithaupt.**

The weakest parts of the ropes forming the cable, as is brought out in the paper, are the sockets. The vertical angles between the backstay and main cable ends are 129° 17′. Subtending this with a chord of 12 ft., which would give a large enough saddle if the cables are fixed to the towers, will give a tangent-to-cable circular arc, the radius of which is 13.99 ft. To bend a $2\frac{1}{8}$-in. wire rope, even if of the stiffest wire used for such rope, over a cylinder of 28 ft. diameter will impair its strength very little; the sockets would still remain considerably the weakest part of the rope. The bending over this cylinder of 28 ft. diameter would not be of the 3-ft. cable, but only of the $2\frac{1}{8}$-in. wire rope, as is obvious from the method of construction.

The three shorter ropes proposed, the heaviest weighing 33 420 lbs., could be more easily transported than a rope in one piece, of 56 980 lbs., reaching from anchorage to anchorage. Transportation of the latter would not be impracticable; if found preferable a thinner rope coming within the desirable weight limit for transportation could be used. The work of paying out one rope from anchorage to anchorage should not be more than that of paying out the three proposed sections, and would be done from a better working base; though it must be borne in mind that the three shorter sections might be reeled off directly from the barges on which they were transported, located at the tower bases.

Friction and the angle of the cable over the tower would in themselves suffice to transmit horizontal stresses from cable to tower, and fastening could easily be further safeguarded by clamps. The proper pulling of the backstay and the main part of the cable against each other would be a matter of adjustment, in position and inclination, of

Mr. Breithaupt. the saddles, and there would remain the same freedom as by the proposed method of giving the backstays the most desirable direction.

There is another objection to the proposed method of fastening the cables to the towers. The cables are at this point increased from a diameter of 3 ft. to one of about 6 ft. The center rise and fall of the cable from stresses and the assumed temperature changes is 10.26 ft. between extremes. This causes material detrimental bending of the cable at the tower fastening, even on a 3-ft. diameter of cable, and is proportionally worse with a 6-ft. diameter.

Making the cables continuous from anchorage to anchorage and using eight saddles, instead of cutting and using the proposed connections, gives an approximate difference in parts, weight and cost, as follows:

4 048 sockets.................	145 728	lbs. at 10 cents	$14 572 80
Details at tops of towers	7 271 376		
Less eight saddles and connections, say..................	600 000		
	6 671 376	lbs. at 5 cents =	$333 568 80
			$348 141 60

The proposed towers do not require longitudinal stiffness, and would be lighter with a narrower longitudinal base, the material saving being in the bracing. In that case the weight would, however, have to be spread again to cover the required foundation area. A continuation of the idea of fastening cables to towers would suggest using rocker bents for the towers. Such bents could be proportioned for direct pressure only, and be much lighter than towers of the ordinary form. They would not be practicable for towers of the proposed magnitude; but for shorter spans, with towers of not over 175 ft., they would afford very considerable advantages.

On the whole for towers approaching the proposed magnitude the advantage remains with the customary arrangement of movable saddles if the towers are given a wide enough base longitudinally to ensure their stiffness in that direction.

CORRESPONDENCE.

GUSTAV LINDENTHAL, M. Am. Soc. C. E.—Although the suspension type was used for the earliest iron bridges, it was the latest for which correct theories of calculation as regards strains from moving loads were developed. The dimensioning of the cables, towers and anchorages was always a simple matter, and, to judge from the records of older suspension bridges, was correctly done in the little light the early engineers had on the strength of iron. Before the era of railroad building, heavy concentrated loads did not need to be considered, being easily avoidable by street regulations. Mr. Lindenthal.

The suspension type for railroad loads was a different problem. Only two suspension bridges for railroads were ever built; one on the braced double-chain system by Engineer Schnirch over the Danube Canal in Vienna in 1860, replaced in 1886 by an iron arch bridge; and the other, the famous wire-cable bridge over the Niagara River, by John A. Roebling, in 1854, being replaced now, also, with an arch bridge.

At the time of their construction no reliable theory of the different systems of stiffening existed, and it is not surprising that both bridges proved inadequate for the concentrated loads of trains, which are growing heavier every year. In spite of the insufficient stiffening, both bridges showed up remarkably well, and together with the Brooklyn suspension bridge, which, although not adapted for railroads, is nevertheless subject to the concentrated load of 100-ton passenger trains meeting on two tracks, augured favorably for the suitability of the suspension type for long railroad spans, when properly designed in the light of modern theory.

As the paper deals with a span corresponding to the dimensions required for a railroad bridge over the North River, for which the writer made the first studies and plans, beginning some twelve years ago, and when a span of 3 000 ft. was looked upon as impracticable, even by some bridge engineers, he read with a special interest the author's views on the subject.

The paper treats of a stiffened suspension bridge in its simplest form; that is, of a combination of cables and stiffening trusses for the middle span, with the backstays of the shore spans carrying only their own weight. The main span is affected only by the loads occurring on it, and the calculations are thus not complicated by loads suspended from the backstays. The bridge would be sufficient for six railroad tracks.

The author invites special attention to two features. One is the continuous stiffening trusses extending beyond the towers, the other his method of cable construction.

Mr. Lindenthal.

The first is the most commendable feature of the design. It would look better, however, if the trusses were extended to the anchorages. The possible turning of the tracks into a curve on the New York side before they reach the towers may explain the unsightly arrangement of one end of the design, but there appears no need for repeating it at the other end of the bridge for the sake of symmetry.

The special merit of the continuous stiffening girder consists in the shifting of the points of contra-flexure under the action of the moving load. It is a prevention of and a protection against localized bending strains in the cables, and a great advantage over the center-hinged girder, which exposes the cable to sharp and continuous bending strains at the center.

The continuity of the girders through the towers is also a good feature, contributive to greater stiffness. It protects the cables near the towers against the effects of sudden loading from fast trains better than girders not continuous through the towers.

Nothing of any account would be gained, theoretically or practically, in varying the sections of the chords or of the web members for such a girder as in the design, and the author chose wisely in adopting a uniform section for the suspended part throughout.

There is no single quantitative law for the bending and shearing strains in continuous stiffening trusses. For such having the heaviest chord sections or greatest height, or both, at the middle, the deflections will be different under the same moving loads as for trusses having them at the quarters. For different assumptions of height or sections for the trusses, there will result differently distributed deflections. While the preliminary calculations for working out, or balancing, the design in the rough, as it were, are simple enough, the final computations, even with the facility of uniform moments of flexure, are sufficiently involved to make the complication from unnecessary variations of section and height undesirable. They offer no compensating advantages.

The writer agrees with the author that the middle hinge in a stiffening truss does not have all the good points claimed for it. The simpler statical calculation for this kind of truss is no advantage, since equal reliability is possible for the more intricate calculations of the continuous girder; they concern the engineer alone, and cannot be counted an objection. The necessity of carrying the top wind bracing down to the lower wind truss at the middle hinge is a grave objection; and as regards the supposed freedom from temperature strains, it is a myth. Theoretically, as well as practically, the hinged stiffening girder requires more material than the continuous girder for equal unit strains. Making the girders high would in both systems lighten the chords and show an apparent saving, were it possible to ignore the temperature effects. They can, however, be no

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more ignored in hinged than in continuous girders. For equal height of trusses, the moment areas from temperature strains are the same in both systems, although differently distributed. Prof. Melan, of Austria, a leading mathematical authority on suspension bridges, and an expert engineer whose judgment the writer considers of weight, expressed himself likewise in favor of the girder without middle hinge in a conversation on this subject. For an important city bridge, the ungainly appearance, particularly of high center-hinged girders, should alone be a decisive objection, and it is merely a question of time when this kind of girder will be classed with the Bollman, Post and other obsolete forms of trusses, which likewise were once thought to have special merits of their own.

The so-called cantilever feature of the stiffening girder produces a considerable saving in weight as compared with the ordinary continuous girder. An approximate calculation shows that for the latter the excess load absorbed by the cable would be only 7% instead of 9.4%, as in the author's design. The simple continuous girder would need to be 22% higher and 8% heavier between towers, including due allowance for the chords tapering from the quarters towards the towers. The cables would be heavier by 3%, and the weight per lineal foot would be 40 600 lbs. instead of 39 000 lbs. The cost of the metallic part of the bridge alone would be increased by $300 000 over the author's design, which in this feature, therefore, possesses also the merit of economy over the simple continuous girder, and still more so over the center-hinged girder.

It is not quite clear why the ratio of dip (using this word for the deflection or versed sine, improperly so called, of the cable) to span was chosen as one to eight, possibly for the sake of high, flexible towers. Flexibility is sometimes an advantage, but great metallic height is a disadvantage. The larger amount of elastic contraction in the towers from the pressure of moving loads, together with the greater sagging and stretch in the longer backstays, causes the tower tops to swing out towards the main span more than with shorter towers; the greater height is also more expensive. It is doubtful whether an economical advantage is gained over a flatter catenary, which would look better.

Flatter catenaries have greater initial rigidity, as the writer would term it, namely, a larger proportion of live load is required to distort the cables. The effect of it, like that of cradled cables on lateral stability, is much more noticeable and beneficial in practice than mere theoretical considerations would indicate.

The excess live load on the loaded half, deflecting 3.5 ft. at the quarter, would increase from 9.4% of the dead load (with the first suspender 200 ft. from the tower) to 11.3% for a dip of one-tenth the span, and to 13.5% for a dip of one-twelfth the span. Thus, assuming for

Mr. Lindenthal. easier comparison the dead load (39 000 lbs. per lineal foot) carried by the cables to remain the same for different ratios of dip to span, the excess load, which in the design takes 3 675 lbs. out of 12 000 lbs. and represents 30.6% of the live load, would take 4 400 lbs., or 36.7% of the live load for a dip of one-tenth the span, and for a dip of one-twelfth it would take 5 260 lbs., or 43.8% of the live load, all for the same deflection of 3.5 ft. at the quarter point. The stiffening trusses would therefore become much lighter, although the cables would become heavier.

The length of span itself has no influence on the percentage of excess load as expressed per lineal foot, *i. e.*, if the span were 1 000 ft. instead of 3 200 ft. (or, virtually, 2 800 ft., as in the author's design), the percentage of excess load would be the same, provided the dead load, the dip of the cable and the deflection at the quarter were the same; but when the length of span is given, the proportion of live load for which the stiffening girder must be dimensioned will be smaller as the cable curve gets flatter, and as the deflection in the quarter is allowed to be greater.

There will be, then, for a given span and live load, and for a given deflection at the quarters, some combination of dip of cable and height of stiffening truss which will result in a design requiring the least amount of metal for the superstructure and towers. Formulas deduced for that purpose are, however, of little practical value, since with the different prices for cables, towers, stiffening trusses, floor construction and masonry and with different details, it is not always the lightest structure which is the most economical. In any given case the best combination will be obtained by the preliminary calculations of a few test designs, and it will then be found somewhat a matter of give and take as regards the different elements affecting the design.

Inasmuch as small variations in the deflection at the quarter will produce very sensible differences in the weight of the stiffening trusses, cables and towers, it should not be made smaller than necessary. In ordinary railroad bridges a deflection equal to 0.001 of the span is considered no impediment to the fastest trains. Indeed, in most of the cantilever bridges built, the deflections are much greater. The fastest trains are some passenger and express trains, hardly exceeding 1 500 lbs. per lineal foot and 600 ft. in length. Assuming such trains on six tracks to meet on the half span of 1 400 ft., there would be 2 700 tons more or less distributed. It is all sufficient to assume that the six passenger trains, running at 50 miles per hour, should cause no greater deflection than 0.001 of the half span, equal to 1.4 ft. The steepest change of grade would be 0.04% for a length of 200 ft. On this assumption, the 8 400 tons maximum of the author would produce a deflection of 4.35 ft. The change of grade would be 1.24% for a short distance, an entirely permissible grade. The load of 8 400 tons

may never meet upon the half span, as it would be an extreme case, while six fast and heavy passenger trains may not meet once in a year with the densest traffic. Mr. Lindenthal.

As mentioned by the author, the dead load could be reduced if the allowable deflection at the quarter were increased. An increase from 3.5 to 4.35 ft. would require an excess load of 11.2 instead of 9.4 per cent. The stiffening girder would be lighter, and slightly less in height for the same unit stresses. The dead load would be reduced from 39 000 lbs. to 37 800 lbs., with all the corresponding saving in cables, towers, etc., from the smaller dead load; that is, for the same dip of cable as in the author's design, one-eighth the span.

If a combination of stiffening truss with a flatter catenary would cost no more, the latter would be preferable. The saving of steel in the towers, for the same dead load of the main span, would be considerable. It would amount to 80 ft. in height for a dip of one-tenth the span, equal to about 7 840 000 lbs. and to 133 ft. in height, or to 3 000 000 lbs. for a dip of one-twelfth the span. Against this saving would be, as a set-off, the increase in the weight of the backstays and in the size of anchorage.

The writer is not prepared to say, without a more extensive investigation than a general discussion would warrant, whether the saving in the weight of the towers and stiffening trusses at 4 cents per pound would balance the increase in weight in the cables and backstays at 7 cents per pound, plus the greater expense of the anchorages. This much can be said, however, that in all cases where the material in the cables is as cheap or cheaper per pound than that in the stiffening trusses and towers, as would happen with eye-bars suitable for shorter spans, there will be a decided saving in cost for a flatter catenary. Almost all the older suspension bridges were built with flat catenaries of one-twelfth to one-fifteenth the span, and to this condition their comparative stiffness in the absence of any adequate stiffening device must be ascribed. It was the result of sound engineering intuition, as even Prof. Rankine's theory, the first one published, required the same weight and strength of stiffening truss for steep and for flat catenaries alike.

The larger temperature strains in stiffening trusses caused by flatter catenaries are not a serious matter, if the former be not made unnecessarily high.

Trusses of hard steel will need to be higher than the equivalent trusses of soft steel for same deflections. The height should, however, not be greater for continuous girders than one twenty-seventh of the span, because in higher girders the influence of the web upon the deflections could no longer be neglected. The short reaches between points of contra-flexure for certain positions of the moving load would, in higher trusses, offer less than the theoretical moment of resistance,

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measured by the chords alone, and the actual deflections would be greater than the calculated ones. A very long or a very heavy span can have low stiffening trusses for the reasons stated, but with the span or dead load becoming smaller, the stiffening trusses will, for the same degree of stiffness, have to be made higher proportionately to the span.

The height chosen by the author in the provisional manner he describes is, therefore, a proper height, and another good feature of his stiffening truss is the riveted connections, with web members riveted up also at their intersections. It greatly helps to diffuse the bending strains throughout the frame, and the so-called secondary strains at all the connections and intersections are a decided gain on the side not only of greater stiffness, but of greater safety, contrary to the received theoretical considerations on this subject.

It is not necessary to combine the bending strains from live load with those from wind pressure and temperature for proportioning the chords, as advised by some engineers, because all the maximum strains can never occur together in such a large structure.

No trains will crowd upon the bridge during a gale strong enough to upset the cars. It will always be prudent to combine the different maxima for aiding the judgment, and keeping on the safe side, as the author has done; but no unbending rule for dimensioning would be justified for conditions of extremely rare occurrence.

In view of the rules prevailing for the dimensioning of bridge members subject to alternate tension and compression, the author seems to consider 20 000 lbs. per square inch a large concession, equal to 40 000 lbs. between the extremes of positive and negative, in 60 000 to 65 000-lb. steel. The writer is of the opinion that for the very rare maxima, the limit can be made higher with this steel, and still higher with hard steel. The usual formulas for alternating strains are precautionary rather than correct, and, in the absence of more accurate knowledge, perhaps justified for short spans and fast trains, but in long spans, where the reversals take place slowly, they are wasteful. It should make a difference whether the alternations take place at the rate of six per second or six times a century; whether they are in the form of shock, or in the nature of gradual or cumulative strains. There is, as yet, not sufficient light on that question.

As regards the suitability of hard steel for members subject to alternate tension and compression, the conclusions of Sir Benjamin Baker are perhaps the latest on this subject. He made tests for the Forth Bridge with spindles weighted at the ends, from which he believed that although hard steel endures repeated bending under given absolute stress better than wrought iron and soft steel, yet it excels them much less in alternating tensile and compressive strength and elastic limit. This, to the writer's mind, is a conclusion not fully

justified by his tests. They all seem to have been made, as far as any record shows to the contrary, at the same rate of revolutions or alternations per time unit. Transmission of stress in steel or iron requires a certain time, like sound or electricity. How much time is not definitely known ; but it may reasonably be assumed that a small dislocation of the molecules requires less time than a large dislocation, all within the limits of elasticity of the material. An oscillation of stress, as in hard steel, from 46 500 lbs. tension to 46 500 lbs. compression, that is, 93 000 lbs. between extremes, as was the case in the tests mentioned, will require more time than an oscillation, as in soft steel, from 26 000 lbs. tension to 26 000 lbs. compression, or 52 000 lbs. between extremes. The modulus of elasticity being nearly the same in hard and soft steels, the stress in the hard steel oscillated, as it were, through an arc 1.8 times longer than that for soft steel. Mr. Lindenthal.

The spindles of hard steel should have revolved slower in proportion, as it required proportionally more time for recovery from stress than did the soft steel at the same rate of recovery. The tests would then have been on the same basis for comparison. For both steels the proportion of ultimate stress to the calculated working stress, called by Sir Benjamin Baker the "factor *A*," was the same. As the rate of revolutions was alike for both steels, the harder steel was comparatively overstrained and likely to give out first, and it did. There is no means of judging whether the hard and the soft steel were not both overstrained. Alternations at slower and faster speeds and with same stresses would be necessary for comparison. Additional tests, systematically made, are needed on this question before a correct judgment can be arrived at. There is, however, nothing in the nature of hard steel *per se* which would make it a less enduring material than soft steel for the slow and gradual alternations of stress in a stiffening truss, which may never have to resist the maxima of the calculations.

The time interval between the alternations would be about 20 seconds for very fast trains at 50 miles per hour. The maxima would require six heavy freight trains abreast at that speed, weighing 8 400 tons. As this mode of loading is out of the question, and considering that the stiffening truss is not as vital to the safety of the bridge as the cables or towers, the writer is of the opinion that 95 000 to 100 000-lb. steel, which is also remarkably tough, as proved in wire drawing, with an elastic limit of 56 000 to 60 000 lbs., is the preferable material, and that 35 000 lbs. tension or compression per square inch is an entirely permissible stress for the chords, from combined live load and temperature maxima, and two-thirds that stress for the web members.

With hard steel the weight of the stiffening trusses in the design could be reduced from 14 340 lbs. to 8 400 lbs. per lineal foot of bridge, the dead load, including cables, from 39 000 lbs. to 32 000 lbs. per lineal foot, and the cost of the bridge would be less by $1 400 000. This is

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for the same deflections, 3.5 ft. at quarter, as are assumed by the author. For the greater deflection of 4.35 ft. the cost would be reduced $460 000 further.

The shop work for the heavy sections, in which all rivet holes must be drilled, will be no more expensive in hard steel than for medium or soft steel.

The reasons given for the use of nickel steel in stiffening trusses do not seem to warrant the large extra expense. The quality of toughness does not come into action in the chords of a stiffening truss, but the quality of elasticity does. Hard steel is the more elastic material. The range of the elastic limit between extremes in hard steel is over 120 000 lbs., against only 72 000 lbs. in medium steel. If the latter were preferred for the greater resulting dead weight and greater inertia of mass in the bridge, then the same advantage, if it be any, can be obtained more cheaply with lighter trusses of hard steel, and with stone ballast at $1 per ton in place of the $80 per ton for steel.

The suitability of hard steel is proved by the arches of the St. Louis Bridge, which for 22 years have been strained nearly 30 000 lbs. in compression, and 20 000 lbs. in tension, per square inch alternately many times a day, without showing the slightest defect. Much better and more uniform steel than that can now be had at a low price. The similar daily strains in the stiffening trusses of hard steel would be 8 000 lbs. tension or compression.

No severer test of hard steel can be had than that of steel rails, which are subject to continuous reversion of strains and to the shock and peening effect of rolling loads.

If nickel steel can be had cheap enough, it would be the material of all others for the cables, for which the quality of toughness is indispensable.

For the thin sections of the floor construction, the usual soft steel is preferable, less for its supposed toughness than for the cheaper shop manipulation of punching and reaming, instead of drilling, the rivet holes.

In the proportioning of the wind bracing, for which the chords of the stiffening trusses act also as the wind chords, the author seems to assume that the lower and upper wind trusses would act together, but this is doubtful. In the absence of adequate cross-bracing one wind truss may deflect sideways more than the other. The maximum side deflection is given as 8.75 ft. One wind truss may deflect 3 ft., the other only 2 ft. A wrenching of the web members is liable to occur where the middle strut is attached. The same wrenching will happen when one stiffening truss deflects vertically more than the other. It would seem better to have no cross-bracing at all than the one proposed. A strong cross-bracing is an advantage, if so designed that the wind trusses

must deflect together, and so that it will not injuriously affect the web members. Mr. Lindenthal.

The relieving of the towers from transverse strains by spreading the backstays would seem a somewhat costly device where land is expensive, and it is not necessary for stability. On the contrary, a strong cross-bracing between the towers is conducive to greater lateral stability. The towers should stand on straight lines at right angles to the axis of the bridge. The backstays can be spread from them also, if desired.

The fact seems to be that the spreading backstays and twisted towers are for the accommodation of the cable connections on top of towers, which is a point that should count against them. The towers should stand at right angles to the axis of the bridge, because the tower tops must deflect in vertical planes parallel to this axis. They cannot deflect, theoretically, in the planes of the spreading backstays with the brace between their tops. It is the more rational arrangement, although it would require an entirely different kind of top bracing, the extra expense of which would be insignificant. The unsightliness of large towers, twisted with reference to each other for so trivial a reason, would hardly appear excusable to the beholder without an explanation which the odd appearance of the structure itself would not suggest. The base of the Eiffel tower furnishes a sample of construction for meeting temperature strains from deep cross-bracing.

No practical advantage results from having the cradled cables further apart on the top of the towers than at the middle of the span. On the contrary, it should be avoided. Should the wind trusses deflect sideways 8.75 ft., given as an extreme by the author, the tower posts and outer cables on the wind side and the inner cable and tower posts on the lee side would have to sustain a larger proportion of load, amounting to an average for the entire length of 4% of the suspended dead load. A better arrangement would be to have the two cables in parallel cradled planes, or one above the other; or, better still, to have one with one-eighth or one-tenth deflection and the other below with one-twelfth deflection in the same cradled plane. It can easily be arranged for both cables to bear accurately one-half or any other fixed proportion of the load at all times, with the great advantage added that cables of different deflections neutralize the tendency to vibration or tremors from moving loads, as distinguished from deflections, a useful office, which in the Brooklyn Bridge, for instance, is incidentally fulfilled by a network of stays.

The most remarkable feature, and the most important also in the author's judgment, is the system of cables composed of coiled-wire ropes, selected on account of readier erection. The ropes are cut over the towers and fastened to them with sockets. The vital importance of this detail to the safety of the structure invites special attention.

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In all tension members for bridges it is held to be essential that on testing they should pull apart between the fastenings, in eye-bars between the eyes, and in rods with upset screw-ends in the body of the bar. In a large suspension bridge, the case of all cases, the same rule ought to hold true, if possible, for the cables. The writer believes with the author that no socket fastening can be made which will break a wire rope between the fastenings. Certain mechanical reasons would at least so indicate, and they are corroborated by the fact that all attempts in that respect have so far failed. Until they succeed, however, cables of wire ropes, having socket fastenings on the towers, should not be thought of for such an important structure. The supposed readier erection will not offset the great disadvantage of having the weakest part of the cable just where it ought to be strongest.

The method proposed for connecting the cables to and on the towers is open to grave objections.

In the first place it will be noticed that the ropes would spread out, beginning at a point 50 ft. from the tower, where the cable would have a diameter of 36 ins. to a diameter of about 7 ft. on the towers. The ropes, supposed to be adjusted to equal tension for a middle temperature, are ordinarily compacted into the round cable form before the superstructure is suspended from them, and it is so assumed by the author in this case. The ropes must hang in vertical planes. In that position they will not all have the same length, because of the twisted towers and the twisted end connections. Each rope will have a different length, which must be adjusted on the towers, so all the ropes shall be strained alike; otherwise, they would have different deflections and could not be compacted. When compacted, the friction will prevent the ropes from sliding past each other, a condition which contributes to stiffness, and is one of the advantages sought in a compacted cable. It is not material, whether the compacting is by means of wire wrapping or by means of clamps, additional or not, to those for the suspenders.

In a compacted cable, having near the towers an enlarged diameter, as in this design, inequality of strains in the ropes from that cause is unavoidable. The upper ropes having to carry part of the weight of the lower ropes will be strained more than the lower ones. It may be taken for the present as a negligible inequality. But the uneven tension in the ropes near the towers caused by swinging the cables from the vertical into the cradled position, and the uneven tension caused by the vertical deflections of the cables, are not negligible quantities. The uneven tension from cradling cannot be prevented by pin bearings set at right angles to the horizontal projection of the tangent of the cradled cables, when, after all, they are to be erected and compacted in vertical planes. The inequality in the tension of the ropes from cradling will depend upon how much the friction

in the compacted cables interferes with torsional sliding of the ropes upon each other. At a low estimate, it may require the outer ropes to extend 1.3 ins. in 50 ft. near the towers in the outside cable. The resulting extra strain from the bending of the cables would be 58 000 lbs. per square inch in the outer ropes, or more than enough to relieve the ropes on the opposite side of cable for 50 ft. from any strain at all. This is on the supposition that the cables have been compacted in vertical planes and for their entire length, except 50 ft. from each tower. Mr. Lindenthal.

Turning for a moment to the vertical deflection, the weight of the superstructure will be found to cause the cables to deflect 10 ft. Taking the case of a hot summer day, when the cables would drop another 3 ft. at the center, combined with heavy trains on the half span, producing at the quarter a further deflection of 1 ft., it will be found that by reason of the change of angle formed by the tangent to the cable curve and the tower, the upper ropes would stretch in 50 ft. as much as ¾ in. more than the ropes in the middle of the cable. Again, on the assumption that the cables are compacted for their entire length except 50 ft. from each tower, the excess strain on them would reach 33 000 lbs. per square inch, or more for greater deflections in the quarter. The ropes on the opposite side of the cable would be strained 33 000 lbs. per square inch less. A diminution, or possibly a reversal, of these excess strains, but much smaller in amount, would take place in the upper and lower ropes on very cold days, at the end of the unloaded half of the cable.

While these excess strains, under the conditions named, when added to the strains from dead and live load, would possibly yet be within the elastic limit of wire ropes, they indicate how liable to overstraining the cables as proposed would be at the point of their greatest weakness, namely, in the socket fastenings at the towers.

A diffusion and great diminution of the unequal tension in the ropes could be effected by pulling the cables into their cradled position at the same time that they are compacted, commencing at the middle of the span, and also at the same time suspending the floor, commencing at the middle and proceeding symmetrically towards both towers. As a further precaution, the cables should not be compacted between the tower and the first suspender, 200 ft. away. This method of erection is not contemplated in the design of the author; but if not done, it may happen that over 10% of the ropes at the towers would be strained to nearly two-thirds their ultimate strength, and only a small number in the core of the cable would be free from excess strains.

It might be urged that the same objection would apply to cables continuous over the towers. So it would, if not guarded against. It is entirely practicable with continuous cables to reduce the excess

Mr. Lindenthal. strains from bending at the towers to a small and negligible amount, even with very large cables. The two cables of 36 ins. diameter could very well be combined into one of 50 ins. diameter. It is not possible, however, with cables cut over the towers.

The cradling of the cables is too valuable a feature to omit on account of bending strains at the towers, which can be fully met for the largest cables, notwithstanding that they be compacted when hanging in vertical planes. Special precautions in the erection of wire cables have not appeared to be necessary before, because the largest so far made were those for the Brooklyn Bridge, 15¾ ins. in diameter, and the bending strains in them at the towers do not exceed 3% of their ultimate strength, as against the possible 18% to 30% under the same conditions for the cables fastened to the towers and erected on the plan proposed by the author.

The second important objection to the author's plan is that the cable connection on the towers is itself not without serious uncertainties. During the erection of the cables only the lower tie plates can be in place. They are arranged to take up in each set the entire pull of one cable from its own weight. This pull is 8 400 000 lbs. per cable. The upper and lower ties are supposed to act together in resisting the maximum pull of 40 000 000 lbs. in the inverse proportion of the distances of their respective center lines of tension from that of the cable, which is about as 2 ft. 8 ins. to 7 ft. 6 ins. on the scale of the detail drawing.

The pull in the lower ties from the cable alone, 4 100 lbs. per square inch of gross section of 2 052 sq. ins., will extend them (in 17 ft. 6 ins., for $E = 28\ 000\ 000$) 0.03 in., which is about twice the least necessary amount of play of a 5-in. pin in the pinhole for purposes of erection. After the upper ties are inserted, and before the play in the pinholes at both their ends can be taken up, the lower ties must further extend twice 0.03 in., which means that they will take up 16 800 000 lbs. more, or a total of 25 200 000 lbs., of tension before the upper ties can come into action. The result would be that of the ultimate 40 000 000 lbs. tension, the lower ties would take up 35 870 000 lbs. and the upper only 4 130 000 lbs., instead of the 29 508 209 lbs. and 10 494 800 lbs. respectively given on detail drawing.

The strength of the nine-bolt connection, set in diamond form in the lower ties, cannot be accurately estimated. All that can be said is that it is probably better than the square form; but even this may be disputed by some engineers, from recent tests made for purposes of comparison with riveted connections where the additional holding force of friction occurs. In the bolt connection it would be absent. To be on the safe side, it will be best to deduct three pinholes from the gross section of 2 052 sq. ins., thus giving a net section for the lower ties of 1 410.75 sq. ins. For a play in the pinhole of 0.027 in.,

which for first-class workmanship is yet a usual allowance for 5-in. pins, the maximum pull of 40 000 000 lbs. would be taken up entirely by the lower ties. The upper ties would then be superfluous. Mr. Lindenthal.

With the most favorable conditions and highest class of machine work, the stress per square inch from 35 870 000 lbs. could therefore be confined to 17 500 lbs. for the gross section, and possibly to 25 400 lbs. for the net section. The latter cannot be regarded as a permissible stress for such a vital connection, which is further subject to large but indefinite bending strains from the holding-down force of the four vertical ties, and from the unequal tension in the cable ropes before mentioned.

All these strains are very large, and slight dislocations, as may be seen, will affect their distribution, so as to produce serious overstraining, amounting to unsafety of some parts.

A remedy might suggest itself in the warming of the upper ties, perhaps with steam pipes, for lengthening them when inserting the three pins at one end, which all must go in at the same time, if they are to resist equal amounts of tension. But whoever has tried the operation with only six eye-bars and one pin will not advisedly propose it for nineteen bars and three pins as an erection device, which, if perchance successful, would introduce obviously only new uncertainties. The only safe way would be the erection of the upper and lower ties at the same time, but that would make the erection of the socketed ropes impossible.

What are, then, the supposed advantages of cutting the ropes over the towers and fastening them there? The author mentions three.

The ropes can be made at the shop, adjusted there to exact length, readily transported to bridge site, and then erected in the least possible time.

Next, avoidance of the decided turns required over the tower saddles, and the readier use, therefore, of a strong stiff wire.

Last, the possibility of using steeper, and, therefore, heavier, backstays to anchorages nearer the towers if necessary, independent of the cable section in the main span.

As regards the first point, the same advantage, in a greater degree, can be had with ropes continuous over the towers, and reaching in one length from anchorage to anchorage, thus avoiding on the towers at least four weak places. The length and weight of such ropes would not be too large for transportation. The adjustment at the shop would be worthless unless the rope is put under sustained tension equal at least to that it would have in the bridge from dead load alone. It can be most easily done in both cases with the ropes hanging on the towers.

The method of erecting the cables on iron horses, supported on temporary ropes with men standing by, and putting each rope into

Mr. Lindenthal. its exact position, can be used equally well for cables continuous over the towers.

Making the wires of one length from tower to tower would be no particular advantage. The splices of, say, fifty wires composing a coiled rope can readily be so distributed, that in any part its strength would be diminished not more than 2%, which is less than the variations occurring in the strength of the wire itself. The friction with which a wire is held in a rope, or even in a wrapped straight-wire cable, is so great in the length of a few feet that it could not be pulled out without breaking.

On the second point, it may be observed that for continuous ropes the flat curve over the tower saddles can be of no possible harm. The wire ropes must be in any event transported, wound around drums of smaller diameter than the saddles on the towers would have. The author himself proposes to use the same ropes for suspenders bent over cable saddles only 36 ins. in diameter.

There is no necessity for preserving the round section of very large cables over the towers. The better plan is to gradually change from the round to a rectangular flat section, which would afford a better bearing on the saddles. The elaboration of such a detail, designed by the writer for another occasion, is here not necessary.

The third point, of separate backstays for steeper inclinations and heavier sections than in the main span, the writer had occasion to consider in connection with an important bridge in Europe on which he was consulted. He is prepared to say, that the increase of section in the backstays, consisting of wire ropes or straight wire, is entirely practicable, without special fastenings on the towers, and without disturbing the continuity of wire ropes or straight wires over the towers.

The precautions in the erection of cables over flexible iron towers (and all towers are flexible) will be necessary not less with cables cut over the towers than with continuous cables, which can likewise have a fixed bearing on them. Care will have to be taken in both cases that under the full load of the superstructure the towers shall have no bending strains in them for that neutral position.

Cables of wire ropes do not offer sufficient advantages to make them preferable to cables composed of strands of parallel wires, the fastening of which to pins by means of looped ends around shoes has been found by experience to be a safe and convenient detail, and a better one cannot be devised. The slow work in spinning the cables for the Brooklyn Bridge is no criterion for similar work elsewhere; it can be done much faster, and it is believed at a price 25% below the price of the coiled wire ropes.

The suspender saddles, as shown on the design, would not be secure against slipping on the steep part of the cable, if the writer may trust

his experience with similar and proportionally stronger clamp details on wire cables only 6½ ins. in diameter. On larger cables, the difficulty of getting a tight grip on the cables is, of course, much greater. Mr. Lindenthal.

It will sometimes be an advantage to continue the stiffening trusses to the anchorages and suspend them from the backstays. The saving of the foundations for intermediate piers between towers and anchorages may more than pay for the extra cost of suspended trusses. The bridge receives not only a better appearance, but there is a gain in rigidity for the main span, provided such an arrangement is not forestalled by local conditions.

It is gratifying to the writer, that the author has arrived at the same general view he held and promulgated long ago, namely, "that a great suspension bridge, which would be well adapted to railroad service, would involve no insurmountable difficulties of construction," and at an estimated cost, the total of which does not greatly differ from that of the writer. That he does not agree, however, with all the views of the author, this discussion sufficiently indicates.

The author was also preceded in his studies by two commissions of engineers investigating the same subject. One of the commissions, consisting of five eminent expert engineers, including the author, arrived at conclusions and estimates more than ordinarily conservative, naturally so in view of the novelty and importance of the problem. The amount of study and labor within the short time allowed to the commission must be designated by those understanding the scope and difficulty of the work as likewise more than ordinary and thorough.

The work of the second commission, consisting of three officers of the Corps of Engineers of the United States Army, one of whom was chairman of both commissions, covered a broader field, and came nearer the truth as regards quantities and cost. Their report contains an original and comprehensive discussion of the leading principles in the construction of suspension bridges and of the derivation of construction weights, and the most complete compilation of data on wind pressure published in any language.

The accumulating literature on the subject of the suitability of the suspension bridge for railroad purposes, and, in fact as the only practicable type for the very longest span, is furnishing valuable data for further study. Indeed, an engineer investigating the problem of a North River bridge for actual construction would not do his full duty if he rested satisfied with the investigation of only one system of suspension bridge, and that the simplest and most obvious in form. The writer had long ago gone further, and spent much labor in examining other systems. It was not only interesting, but necessary, to make comparisons of the different systems, and with the information so collected, to decide upon that system which for the given conditions would answer best.

Mr. Lindenthal.

It may be remarked that Fidler's system of double cables, intersecting at the middle of the span and near the towers, with the particular arrangement shown in plan No. 3 of the published preliminary designs for the Forth Bridge, showed temptingly great advantages. It is very economical, and can also be made architecturally pleasing, but the erection of this system with wire cables presented difficulties which do not occur with the braced double-cable system selected by the writer for his design. It took some study and time before the details were entirely satisfactory; but they are so now. A detailed description of the design would of itself be a long paper, which may not be presented here as a part of this discussion.

Mr. Hildenbrand.

W. Hildenbrand, Esq.—The following remarks are confined to that part of the design which the author considers to be radically different from suspension bridges hitherto built. He would have been more precise in saying, different from large suspension bridges hitherto built, because the same general principles that guided him in his design have been and still are applied to small bridges. When John A. Roebling and his son, Washington A. Roebling, the builders of the first large suspension bridges, abandoned those features and substituted others, it was with the object, according to their opinion, of improving in expediency and economy the methods of cable-making.

There are three distinct features in which the author's cables differ from those of the present large bridges:

First.—The cables are composed of twisted wire ropes in place of straight wire strands.

Second.—The cables are immovably fixed on the towers in place of resting in roller saddles.

Third.—The cables are not continuous, but consist of separate lengths for each span.

The first two features are common to most small bridges; the third feature has not frequently been applied.

Wire-Rope Cables.—The author gives the reasons why cables made of wire ropes have been discarded for large bridges, viz., because wire ropes have proportionally less strength than straight wire, and because no attachments can be made with any reliable uniformity in strength. The difference between the strength of a wire rope and the same section of straight wire has never been expressed in a formula. It is only known that it varies from nothing, which is an exceptional occurrence, to 15%, which is a frequent occurrence. In the wire-rope list of John A. Roebling's Sons Company the strength of the ropes is rated about 12½% lower than what it should be theoretically, and 10% has generally been accepted as a fair average.

The fastening of ropes in sockets is a still greater factor of uncertainty. Ropes can and have been socketed developing the full strength of the rope, but it requires large, clumsy sockets and extreme care and

Mr. Hildenbrand.

skill in putting on the socket, such as may be exercised in experimental cases. It would not be practical to rely on perfect workmanship, but allowance must be made for the greater or lesser skill of the operator.

Under ordinary circumstances and care the socket strength is frequently not over 75% of that of the rope, while a strength of 85% may be considered a good average. This deficiency of 15% added to the above-mentioned loss of 10% makes a total loss of 23½% in the strength of a wire-rope cable as compared with a straight-wire cable. For the sake of argument, the writer will make an extra allowance of 8½% for special care in the manufacture of rope cables and for losses in wire cables on account of kinks and splices in the wire, and call the loss of relative strength between the two kinds of cables but 15%, which is a liberal assumption in favor of rope cables. This will give the following comparative cost for the case of the author's bridge:

58 553 000 lbs. of wire ropes at the minimum price of 5½ cents per pound..............	$3 220 431
58 553 000 lbs. — 15% = 49 770 000 lbs. of wire at a maximum price of 3¾ cents per pound,	1 866 375
Difference	$1 354 056

The writer believes that a saving of $1 350 000 alone in the cable material, not counting the costly work of socketing, would in most cases decide against the adoption of wire ropes, unless there be other and greater advantages connected with the latter to counterbalance this loss.

Fixing the Cables to Metal Towers.—This can be done safely, as the author has demonstrated by calculating the ordinary motions under passing loads and under changes of temperature, and comparing them with the flexibility of the towers. It seems to the writer, however, that this feature has been chosen unfortunately in connection with another feature of the design, viz., the omission of suspenders in the land spans. When there are suspended land spans, the cables are balanced at each stage of the erection, but when the land cable must first balance the empty, and afterwards the loaded, center cable, it is necessary to give the land cable, when first put up, a greater length than it will have in the finished bridge. For the author's bridge, the writer calculated this difference of length to be 3½ ft., or, better expressed, it requires a motion of 3½ ft. on top of the towers to balance the spans before and after being loaded with the dead weight. Is the author prepared to allow this motion to the towers? It would be much simpler to rest the cables in roller saddles, place the latter 3½ ft. out of centers, and let them gradually roll into their final position.

Mr. Hildenbrand.

Of course, the difficulty could be obviated by temporarily supporting the land cables, which is frequently done in small bridges; but this would prove to be an expensive "extra" in a bridge of such large dimensions and where the land cables, fully or partially, cross a deep river.

3. *Discontinuity of the Cable Over Three Spans.*—This is a feature which, naturally, has not often been applied or suggested. Engineers, generally, try to avoid splices and use the longest available lengths of bridge members; hence, when it is just as easy to get such a member, a wire rope or wire strand, in one length, it seems injudicious to cut it deliberately into three pieces and splice them again, unless this be justified by strong reasons and decided advantages.

In the case of the new Danube Bridge at Budapest, where the same feature of socketing the cables on top of the towers was proposed, the designers considered it justified by the difference of tension in the land and river cables which, if made continuous, had necessitated a considerable waste of cable material. No such reason exists in the author's design, but he states for his reason the two following advantages: that it would shorten the time of erection, by obviating the manufacture and regulation of wire strands, and that it would avoid bending the wires over saddles and end pins.

This second advantage can at once be dismissed, because it is absolutely of no disadvantage for the wire to be bent over saddles and around the end shoes.

The first-claimed advantage looks very plausible, but it is based on an improbability and a misconception of the different phases of wire cable-making.

The improbability consists in the author's belief that 253 wire ropes, each 3 300 ft. long, could be cut to precise lengths and manufactured with such homogeneousness, that all ropes of the same length would retain an equal length if suspended with the same deflection. That this is not absolutely possible needs no proof, because there are no two things alike in the universe, but that it is not even practically probable a few figures will demonstrate.

Each rope is supposed to weigh 10 lbs. per foot, contains 3 sq. ins. of metal, and has a length of about 3 333 ft. if suspended with 400 ft. deflection. The stress in each rope, due to the dead load, will be 70 tons. When the rope is first suspended, its tension will be about 18 tons, or 12 000 lbs. per square inch. The author records the result of some tests, showing that the elongation in 200 ins. of one sample of rope for a tension of 12 000 lbs. per square inch would be 0.1671 in., and in another sample of rope 0.2012 in., being a difference of 0.0341 in. in 200 ins., or of 6.82 ins. $=0.57$ ft., in a length of 3 328 ft. This difference in length would cause a difference in deflection of 0.862 ft. $= 10\frac{3}{8}$ ins. Such a discrepancy in deflection would not be considered a good

Mr. Hildenbrand.

regulation, and this would be the very best that could be accomplished according to the apparently favorable tests quoted by the author. In reality the case would be much worse, because the difference of elongation between two ropes, if measured in a length of 3 000 ft., would be much larger proportionally than if measured in 17 ft., on which the above calculation was based. Considering also the changeable specific gravity of the metal, the difference of temperature between different ropes when cut to length, the impossibility of observing accurately the temperature or of measuring the length, and the difficulty of putting on sockets to occupy a precise length, it will be seen there are many more factors for causing the ropes to hang at different heights, if the variable modulus of elasticity, as proved above, should not be sufficiently convincing. In other words, it would never do to suspend the ropes indiscriminately with blind faith in shop accuracy, but it will be necessary to regulate them relatively. This being admitted, all supposed advantages of socketing are gone, and nothing is left but the disadvantages of inconvenience, loss of strength, loss of time, and useless expense.

The advantage claimed for the rope cable, that ropes can be brought ready made to the bridge site and save the time for spinning a wire strand, is based on the belief that the latter work requires a very long time. This is erroneous; generally, there is more time consumed in regulating a strand than in making it. A wire strand as strong as one of the ropes in the author's cable would consist of ninety No. 6 wires, which could easily be laid up in one day, giving 12½ minutes for a trip of the wire wheel and 12½ minutes for regulating two wires, which is ample. The author believes he could haul three ropes over and put them in place in one day, but according to the experience gained at the Brooklyn Bridge, where a number of ropes were hauled over for temporary purposes, he will do well if he puts one rope in place in two days. He may possibly do a little better than this, but in no case does it appear that he would gain much, if any, time over that needed for making a wire strand of equal strength. On the other hand, the regulating apparatus on top of narrow towers will be much less convenient than if placed on the spacious anchorages. This, and similar other points in cable-making, will become more apparent when "the various special appliances are worked out" to which the author finds it occasionally necessary to refer in his paper.

Taken as a whole, the author's plan could creditably be executed precisely as it stands. There is nothing in the design that could not be sustained theoretically, or be built practically, and the details have been admirably worked through with minuteness and on scientific principles; but from what the writer has tried to show by figures and arguments, he would venture to predict that the cables for the next large suspension bridge will not be constructed on the author's present plan.

Mr. Hildenbrand.

There are other features in the proposed design with which the writer fully agrees. Among these are the omission of suspenders in the land spans, the omission of suspenders for 150 ft. out from the towers, the construction of the stiffening truss as a cantilever for 150 ft. out from the towers, the anchoring of the stiffening truss at the towers, and the continuous extension of the stiffening truss over the land spans. All of these features were adopted by the writer in the reconstruction of the Covington and Cincinnati suspension bridge, which is at present in course of construction. In a few months from now it will practically be demonstrated whether these latter features are judicious applications or not.

Maj. Raymond

C. W. Raymond, M. Am. Soc. C. E.—This paper is of special interest in view of recent investigations with reference to the construction of suspension bridges of very large span. In August, 1894, a board of bridge engineers appointed by the President of the United States* submitted a report upon the length of span, not less than 2 000 ft., which would be safe and practicable for a railroad bridge to be constructed over the Hudson River at New York, in which report the subject of suspension bridges of large span received special consideration. A month later, a board of officers of the Corps of Engineers, appointed under the instructions of the Secretary of War, submitted a report upon the maximum span practicable for suspension bridges. The calculations and investigations of both boards were necessarily approximate in character, owing to the limited time available for the preparation of the reports. The author was a very active and valuable member of the board first mentioned, and the writer, who was a member of both boards, considers it of great importance that this detailed study practically confirms the conclusions of both boards in all essential matters.

As the author points out, the suspension bridge has been generally superseded by truss and cantilever bridges for all except very large spans. The cases requiring suspension bridges will hereafter be rare, and the engineers who design them will not have a practice founded upon extended experience upon which to base their calculations. This fact gives an increased importance to conclusions arrived at by capable and experienced bridge engineers in the few cases studied in detail.

As was found by the Army Board, it is almost impossible to discuss the theory of the suspension bridge satisfactorily in a general way, because the problem is essentially one of details, and the details depend largely upon accidents of location and the relative cost of shopwork and the materials of construction. The method adopted by the author of taking a concrete case and working it out in detail is undoubtedly the only one which can give definite results. The case selected is the one approximately investigated by both the New York

* Under the Act of June 7th, 1894.

Bridge Board and the Army Board; and although the location chosen is higher up the river, it does not differ essentially in length of span from that discussed by Gustav Lindenthal, M. Am. Soc. C. E., in his elaborate and detailed design for the North River Bridge at Twenty-third Street, which design was completed several years before the commencement of any of the investigations previously referred to. Two studies of this problem were made by Mr. W. Hildenbrand for the New York Chamber of Commerce, and are published as appendices to the report of the New York Board. Although these studies were of a preliminary character, they deserve careful consideration on account of their author's connection with the erection of the East River Bridge and his long experience in wire construction. Recently L. L. Buck, M. Am. Soc. C. E., prepared detailed plans for the new bridge across the East River, which will be of great interest, although the span is only 1 600 ft. Maj. Raymond

While it is not possible to make a full comparative study of these different investigations within the limits of this discussion, it may be of interest to point out some of their principal agreements and differences.

Perhaps the most important point in connection with these investigations is the claim which has been made that the suspension bridge differs so much in structural character from other bridges that the rules usually adopted for unit stresses and safety factors require modification in this case. The writer believes that this has been fully stated by Mr. Lindenthal and recognized in his computation, and this view was also adopted by Mr. Hildenbrand in his designs. It was also recognized in the report of the New York Board.

The Army Board devoted considerable attention to this question. It remarked that "the great distinction between the stable equilibrium of a suspension bridge, which cannot break down from the failure of any stiffening member, and the unstable equilibrium of a truss, arch or cantilever bridge, in which the failure of a member may involve the collapse of the entire bridge, ought to receive full recognition in the adoption of unit stresses and safety factors." Again, the Board remarked that "rigidity is in this case of much less importance than it is in most other kinds of bridges; indeed, it may be shown that a certain small flexibility is a positive advantage in suspension bridges;" and still again, "the Board does not doubt that within narrow limits a certain degree of flexibility is an advantage to the bridge. Deflections in a system of stable equilibrium do not impair the safety of the structure as they do in an unstable system like the upright arch, and they may exert a very beneficial influence in modifying the dynamic effects of a rapidly varying live load."

It gives the writer great pleasure to find these views, which were by no means formerly accepted by all competent bridge engineers, fully stated and confirmed in the paper under discussion.

Maj. Raymond. In accordance with these views, the Army Board adopted for the cables a working stress of one-third the static breaking load, and this is a greater working stress than has usually been adopted in the cables of suspension bridges of ordinary span. The same stress was adopted by Mr. Hildenbrand and also by the New York Board for comparative purposes. The Army Board gave in full its reasons for adopting this increased stress. The author has practically adopted the same working stress.

For the stiffening girder the Army Board adopted 15 000 lbs. per square inch for the working stress, while the New York Board limited the stresses due to a moving load to 12 500 lbs., but allowed the stresses from the combined effects of moving load and wind to run up to 22 500 lbs. The Army Board remarks that "the only duty of these girders is to distribute the live load and thus prevent inconvenient deflections. It is not necessary to give them the margin of strength which they would require if they were essential to the stability of the bridge."

Mr. Hildenbrand adopted a working stress of 20 000 lbs. per square inch in his first design, but in his second design reduced the stress to 15 000 lbs. at the request of the New York Board. He believes that 20 000 lbs. will give ample safety because the reverse strains will occur only at long intervals.

The author proposes to use nickel steel in the construction of his stiffening girders and adopts a working stress of 17 000 lbs. per square inch. He remarks that "a suspension bridge must be permitted to change its shape within proper elastic limits, and this change of shape must be made the basis of calculations in proportioning the structure." In the writer's opinion, this simple statement embodies the only rational theory which can be followed in the designing of suspension bridges.

The writer does not know exactly what working stresses were adopted by Mr. Lindenthal for his girders, but an examination of his design leads to the belief that he would not consider the stresses adopted by the Army Board as too great.

As has been before remarked, the suspension bridge problem is essentially one of detail. Perhaps the most important and troublesome detail in the whole problem is the method to be adopted for transferring the stresses and providing for changes of form over the tops of the towers. It is believed that no method has been employed or proposed which is not open to some objections. The author proposes an entirely new system, fastening the ropes forming his cables in sockets at the tops of the towers and taking up changes in the length of the backstays by the motion of the towers alone. This modification, as the author remarks, is really the essential feature of the whole design, and he states the objections to the system and its advantages. There

are two important particulars in which the adoption of this system appears to have modified the general design of the structure—the dip of the cable curve and the character of the vertical bracing. Maj. Raymond.

The author adopts one-eighth of the span for the dip. This was the value adopted by both the New York and Army Boards. The writer believes that a smaller dip would be decidedly more economical, if the fastenings are left out of consideration. A decrease in the dip requires more metal in the cables, but less metal in the stiffening girder. In Mr. Lindenthal's bridge the dip is one-tenth, and Mr. Buck appears to have taken about one-ninth. Mr. Hildenbrand adopted one-tenth in his first, and one-eighth in his second, design. In the author's design an expensive material is used in the girder, and therefore it would seem economical to decrease the weight of the girder and increase the weight of the cables by diminishing the dip. Doubtless the author has adopted the larger dip in order to make his towers sufficiently flexible for the purposes of his cable fastenings.

For vertical bracing, the author adopts the straight continuous stiffening girder. Both of the Boards and Mr. Hildenbrand adopted a straight girder hinged in the middle in order to simplify the calculations. The writer believes that the girder hinged in the middle has no advantage whatever over the continuous girder. Even its supposed advantage in the easy determination of the stresses exists only when the computations are of an approximate character, and vanishes entirely when the temperature stresses in the half arches and the details of the middle hinge are considered. If a straight stiffening girder is employed it is considered beyond question that it should be continuous; if for no other reason, because the portion of the live load taken up by the cable is greater with the continuous than with the middle-hinged girder. Mr. Hildenbrand remarks that the weight of the girder will be reduced nearly 10% by making it continuous. Such a girder is adopted by Mr. Buck in his recent design. Looking at the question, however, from a purely theoretical point of view, and leaving out of consideration the arrangements of the cables at the towers and other details, the writer believes that the best place for the principal vertical bracing is in the suspended arch itself. It is true that the longitudinal girder is needed as a floor stiffener, for the support of the lateral bracing and for side guards; but it is nevertheless believed that the same amount of rigidity could be obtained with less weight of metal if the principal bracing were placed between the suspended cables. Mr. Lindenthal has done this in his design. He combines the two systems (which is perfectly practicable since they deflect with the same rate and law), but apparently requires very little from his longitudinal girder, depending almost entirely upon the bracing of his arch to control the changes in the form of his cable curve. With the method of

Maj. Raymond. cable fastening adopted by the author, the employment of the arch bracing would seem to be impracticable.

There are many other points in this admirable paper which the writer would like to notice if time and space permitted, but these must be left to others. In conclusion it will be of interest to compare the various estimates of cost which have been prepared for a suspension bridge at the locality considered. The estimate of the New York Board for its "Lighter Structure," when corrected to correspond to the "upper location," is about $26 000 000. This estimate, however, was made for comparative purposes, and the Board was properly conservative as regards prices and working stresses. If the cost of structural metalwork were reduced from 4½ cents to 4 cents per pound, and the cost of wirework from 8 to 7 cents per pound, the estimate would become about $24 340 000. Mr. Lindenthal estimates the cost of his bridge at about $21 000 000, but reduces his estimate to about $17 800 000 for this locality. Mr. Hildenbrand's estimate for his second design is about $15 901 000. The Army Board named the sum of $23 000 000 as a reasonable estimate for the cost of constructing a suspension bridge of this length and character. For the design submitted in this paper, which has been investigated with much more care and in much greater detail than any of the others except that of Mr. Lindenthal, the author estimates the cost at $22 500 000.

Mr. Bouscaren. G. Bouscaren, M. Am. Soc. C. E.—The leading features in the author's study of a long-span suspension bridge for railroad purposes are the composition of the cables of ready-made twisted wire ropes instead of straight wires, their discontinuity at the towers and division into main cables and backstays, their fixed attachment to the towers, the particular form of anchorage proposed, and the cantilever shape of the stiffening truss.

The substitution of twisted wire ropes for straight wires raises the question of uniformity of working stress in the different elements of the same cable, which is of first importance. This uniformity is secured to a certainty in the straight wire cables by the method of construction introduced by Roebling, which consists in adjusting every wire to the same deflection. It does not seem that such adjustment can safely be omitted in this case. After all possible care is taken in the delicate operation of measuring every individual rope and the exact distances between centers of towers, and between towers and anchorage connections, possibilities of discrepancies large enough to require a considerable depth of shouldering between the washer plates and sockets will still exist.

The degree of uniformity in the manufacture of the ropes would also affect the uniformity in the working stress of the cables. The favorable results of the tests made at the Watertown arsenal should be confirmed by a large additional number of similar tests before reliable conclu-

sions can be drawn therefrom. In this connection it would be of special interest to note the exact lengths of specimens between shoulders to determine whether an appreciable amount of slip occurs in the sockets within the limit of the working stress. Mr. Bouscaren.

The proposed covering of the cables with jackets made of non-conducting material is a very essential feature in cables of as large diameter as proposed. This jacketing would preserve a sufficient degree of uniformity in the temperature of the inside and outside ropes; it would also add very materially to the protection of the cables against rusting by shedding the rain water, and would prevent internal condensation in a large measure; but it should not be relied upon exclusively for protection, especially if it were to be discontinued over the clamps as proposed. The continuous wire wrapping, which has been so successfully used on the cables of the Brooklyn and Cincinnati bridges, should not be omitted. Before wrapping, all the interstices between the ropes should be filled with a waterproof material neutral in affinity and stable in composition. Outside of the question of overloading, corrosion is the one element which renders uncertain the life of metallic structures. No precaution should be neglected to prevent it in a bridge of such importance, which should possess in the highest degree a character of durability and permanency.

The idea of a fixed connection of the cables at the towers is attractive. It utilizes the stiffness of the towers to lessen the deflection of the span. The principal objection to a fixed connection is the difficulty of obtaining the necessary amplitude of longitudinal motion at the top of the towers during erection without creating excessive bending stresses in the towers. This difficulty can be overcome as suggested by the author, or, preferably, the writer thinks, by meeting the erection stresses with additional metal in the towers. As these stresses are only temporary, an increase of 25% over the normal working units is admissible.

Two objections may be made to the simple and ingenious form of anchorage proposed by the author—first, the temporary lower anchorage might be an expensive affair in the absence of solid rock at a convenient depth; second, the completion of the anchorage masonry for the bearing of the upper casting must be delayed until the completion of the cables, which involves the transmission of considerable pressure through green masonry.

The author's treatment of the question of stiffening is logical and proper. It is quite clear that the long undulations in the cables of such a long span, due to the moving load, are harmless, and that no provision need be made for the distribution of a load smaller than that which would produce an inclination of grade equal to the maximum gradient fixed upon for the convenience of operation. The function of the stiffening truss is, therefore, reduced to the distribu-

Mr. Bouscaren. tion of the difference which may exist between the actual maximum moving load and the load which would produce the maximum gradient. This consideration allows at once a very large economy to be made in the weight of material in the truss.

The utilization of cantilever projections of the shore span to support the stiffening truss has the advantage of securing continuity without the disturbing effect of possible settlements at the towers.

The proposed use of nickel steel in the chords is an innovation in the right direction, like the substitution of steel for iron fifteen years ago. Nickel steel has been used chiefly for armor plates and for special parts of machinery; it is as yet untried on a large scale for structural purposes. The practicability of procuring with desirable promptness 16 000 000 lbs. of this material, possessing a sufficient degree of uniformity in its physical qualities to bear safely a working stress of 40 000 lbs. per square inch, may be questioned, but there are no reasons to suppose that a satisfactory material could not be procured if sufficient time were allowed for experiments. If, as stated by the author, it can be manufactured now into shapes at an advance of only three-quarters of a cent per pound over ordinary structural steel, it could be introduced with economy in the construction of very large truss spans.

The units of working stress used by the author in the different parts of the structure are approximately the same as those adopted by the Board of Engineers in their report upon the New York and New Jersey bridge, but the carrying capacity of his bridge is less than that estimated by the Board, and the prices used in his estimate are lower than those of the Board, which explain the large difference between the total estimates of cost, as given in the two papers.

Mr. Crowell. FOSTER CROWELL, M. Am. Soc. C. E.—In presenting the subject of this admirable paper in the form of a preliminary study of a practical character, the author, it would appear, has invited the frankest criticism and a wide range of suggestions, not only upon the radical and bold departures, or advances, from present recognized features of construction which he advocates, but, in addition, in regard to any other considerations wherein improvement is possible and desirable.

The four chief points of novelty in the design, which are the use of twisted wire ropes in the cables, the fixed attachment of cables to towers, the form of the anchorage connection, and the combination of continuous stiffening truss and its cantilever anchor, together with the various details in connection therewith, have been well discussed. The writer will pass to the importance of providing for the grace and majesty of such a grand bridge, by incorporating in its very structure the forms and lines that shall produce on eye and mind the impressions of a beauty equally grand.

It is commonly but erroneously considered by engineers that utility in a bridge design is all that is necessary to render it pleasing to the eye, or, at best, that the Aristotelian principles of beauty, which were "Order, Symmetry and the Definitive," are all that need to be observed. Even the supreme ugliness of the ungainly Forth Bridge is said to be transformed into beauty when one comprehends the functions of its several parts, but few besides engineers can appreciate those functions, and so, to most observers, it is and always will be one of the ugliest as well as most stupendous structures. Mr. Crowell.

It would seem that the author shares the view that in an engineering structure "handsome is as handsome does," for he is silent in regard to the matter of proportion and form, excepting as called for by the strain sheet or the requirements of the laws of stability; in fact, he even introduces intentional irregularities in the plan and placing of his towers, and treats them as matters of no consequence, although he refers to the fact that the proposed twisting of the towers with reference to each other would be manifest, and concludes that the fact of the towers themselves being built out of square would seldom be observed. The only references the paper contains to the appearance of the structure are as follows: "Fig. 15, showing the general elevation and tower, shows the tower as finished, with a room about 50 ft. square on top, surmounted by ornamental work terminating in a flagstaff," and "the ornamental work on the top of the towers, with provisions for lighting, etc., would cost $100 000." It is the writer's present purpose to call attention to the avoidance of purely æsthetic principles in the author's design, not as a distinctive demerit, but because it is a characteristic lack in most designs of modern metallic bridges, large or small, and to urge that it is of the very first importance in developing a design like this, that those principles should be fully recognized.

By this the writer does not wish to be understood as recommending either mere decoration or ornamentation superadded for effect, but that the forms and lines of the parts of the structure wherein a choice is possible should be both expressive of their function and harmonious with the dominating lines, which, in the case of suspension bridges, are the sweeps of the cables.

Nature abhors a straight line as much as she does a vacuum, and it is by the use of all sorts of orders of curves that the impressions of beauty are produced. When, then, straight-line towers are introduced into the design of a suspension bridge, the principles of beauty are rudely violated, and the fact that the lines of floor and stiffening truss are also curved emphasizes the incongruity to the point of ugliness. Furthermore, the towers are not simply shafts, rising to a great and conspicuous height; they are to support a visible weight; their purpose is plainly revealed. They should, therefore, be so designed in out-

Mr. Crowell. line as to give the right impression of strength continued to the point of support. The cables should not appear to pass through them, as do the cables in the otherwise beautiful bridge in Budapest, but to rest upon them. With the carrying of the cables their function ends, so they should not be carried higher than absolutely necessary, and they should not have finials and other features of ornamentation; especially should they not have flagstaffs. If there are to be observatories on the towers they should be plainly in evidence; all the finish at the top of the towers should be of great simplicity and massiveness. The proportions and lines of the towers should give an expression of repose and superabundant strength combined with lightness. Then, too, they should be of such outline in plan as to eliminate the necessity for being placed unsymmetrically, even at some sacrifice of economy of material and cost.

The position of the rocking bents should be back of the front posts, masked, as far as possible, by the towers, provided this can be done conveniently, which would seem to be the case, for, pleasing as the device, as designed, might prove, viewed as a means to an end, the harmony of the proportions of the towers would be sadly marred.

There is nothing novel in the principle here suggested, apart from its application to bridge design. The value of the curve in architecture and its use for centuries to produce impressions upon the mind of the observer, whether in the outline of a column, or in the proportion of spire or cathedral arch or dome, or in rectifying the crooked effect produced by a straight cornice, need not be cited. One has only to contrast mentally the effect of the Eiffel Tower with the straight line skeleton observatories and electric light towers commonly seen, which have even less beauty than an oil-well derrick. There is, moreover, a utilitarian consideration for employing a curved outline for the suspension bridge tower designed to be flexible under the action of the cables, but that is a refinement that need not here be discussed.

Passing briefly to another feature of the plan, the dropping of the two spans at each end of the structure, *i. e.*, making them deck spans, is an artistic mistake, however sound the reasons for it, in a strictly engineering sense, may be. The through structure principle should be made use of on to the anchorages, with the necessary expansion break at the cantilever end, of course, and should terminate in grand portals, surmounting and dignifying the otherwise ungraceful and characterless anchorages. There would also be a utilitarian advantage in such treatment in securing greater immunity from fire dangers from adjacent buildings, and if the space beneath were to be utilized for fireproof structures, the additional headroom so secured would be valuable. This treatment would allow of a magnificent promenade, also a source of revenue.

The idea of adorning bridges is in itself not a new thing; in fact

there are many examples of the attempts to create beauty by adding ornaments and so-called architectural devices, most of which, by the way, are complete and dismal failures. One of the most conspicuous cases the writer recalls is the disguising of a straight truss as a colonnade with false column bases and capitals and connecting arches of thin cast iron. The bridge is a double-deck structure, and the colonnade is supposed to have the effect of supporting the upper deck, regardless of the fact that the bases of the pseudo columns are resting on air. The effect is, of course, worse than grotesque—it is abominable. Mr. Crowell.

Sometimes an arch bridge of notable span length is utterly devoid of impressiveness and beauty because of the introduction of straight-line connecting spans between the ends of the arch and *terra firma*. Sometimes the effect of an otherwise magnificent arch is completely lost, because it is one of a pair instead of a single one, or one of three.

Ruskin says that the greatest art which the world has produced is that which is fitted for a place and subordinated to a purpose, and that all art worthy of the name is "good craftsmanship and work of the fingers, joined with good emotion and work of the heart; that is, the energy of body and soul united." If there be room for the application of this idea in the grand art of engineering, of which the writer thinks there can be no doubt, it will behoove the designers of monumental structures, like the one under discussion, to bring to their aid all the resources of art to produce in the highest attainable degree perfection of form conjointly with mechanical excellence.

J. E. GREINER, M. Am. Soc. C. E.—The author has developed a mechanical device, the object of which is to transfer the strain from the main cable to the backstay, thereby avoiding a continuous cable from anchorage to anchorage. It is hardly probable that any one will deny that the arrangement illustrated is novel, although socketed cables have been proposed before, the most prominent design being in the first-prize plan of the Budapest bridge. Mr. Greiner.

The question arises, will the saving in cost of erection justify the use of such an expensive arrangement, expensive not so much on account of the labor and material used in its construction as on account of the large amount of unavailable material placed in the cables? In the case under discussion, the details do not develop the strength of the members connected, and until experiments demonstrate that a socketed joint can be made strong enough to break the cables, engineers generally will avoid its use. As the capacity of the proposed bridge depends upon the strength of its cables, it may be well to inquire what excess of material must be placed in the cables in order that the sockets may be strong enough for the loads.

The tests quoted by the author show that sockets will, on an average, develop from about 70.4% to 83.1% of the strength of the sample plow steel wires, or from 76.8% to 85.9% of sample special steel wires,

Mr. Greiner. and that, while tests of wires gave an average strength of 172 588 lbs. for special steel and 226 504 lbs. for plow steel, a difference of 31.2% in favor of the latter, the sockets attached to the plow steel cables developed an increased strength over those attached to the special steel of but 20.3 to 26.9 per cent. This indicates that the strength of the socketed cables does not increase in the same ratio as the ultimate strength of the different kinds of sample wires tested, and that the ratio is unfavorable to the socket.

According to the author's calculations, the cables will weigh 57 663 760 lbs. The socketed joints can develop from 70.4% to 85.9% of the strength represented by this weight, consequently there is from 14.1% to 29.6% of the material in the cables practically unavailable. This means that in order to use the proposed sockets it will be necessary to waste from about 8 000 000 to 17 000 000 lbs. of cable worth 7 cents per pound, or from $560 000 to $1 190 000. This is but a small percentage of the total cost; nevertheless, when dealing with millions, a very small percentage represents a large amount of money. If this amount can be saved in the erection, there can remain no objection to the excessive material used, and if the time of erection is lessened, the saving of interest on the large capital invested may result in a decided saving in the total cost of the completed structure.

It will be noticed that the author avoids expansion sockets over top of towers and allows the towers to deflect from the perpendicular. Similar means of taking care of expansion are in common use on a small scale, notably in train sheds. The application of the same principle on such a large scale is perfectly legitimate and appears to the writer as the proper thing to do in all cases of long-span superstructures. It is economical and perfectly practicable.

Other details of the design, namely, the anchorage and accessibility of all parts to inspection, are specially good features. In the writer's judgment, however, it would be best to cover the cables with close-woven wire thickly coated with white lead so as to protect them absolutely against the entrance of moisture. After being thus protected they could be covered by a layer of non-conducting substance which would allow heat from the sun's rays to reach the metal of the cables no faster than it is dissipated through the whole volume of the cables. In an address before the American Association for the Advancement of Science, read by Gustav Lindenthal, M. Am. Soc. C. E., on the "Economic Condition of Long-Span Bridges," it was proposed to use cables of parallel wires, but instead of being wrapped with wire, they would be enclosed in a water-tight steel envelope allowing an air space of about 2 ins. to prevent uneven heating of the wires by the sun. By using some such casing and packing the space with a non-conducting material, the wires could be kept at a nearly uniform temperature.

It has always appeared to the writer that a suspension bridge

designed with stiffening trusses hanging to the main cables has something lacking in its æsthetic as well as mechanical construction. While a cable suspended from tower to tower assumes a beautiful curve and lends an appearance of grace to the bridge in spite of a crude and clumsy suspended stiffening truss, the fact should not be lost sight of that it is the curve which the cable assumes and not the comparatively thin cable which appeals to the æsthetic sense. When analyzing the beauty of a suspension bridge an appearance of bird-cage frailty will be found invariably in that part of the structure between the cable and stiffening truss, a lack of something not found wanting in an arch bridge. This void between the cable and the floor can be filled admirably by placing one cable over the other and bracing them together as proposed by Mr. Lindenthal in his design for the North River bridge. When arranged in this manner the cables will assume the same graceful curve, and will have the appearance of strength and rigidity where it is natural to expect it. If the cables of a suspension bridge can be utilized for chords of stiffening trusses, Mr. Greiner.

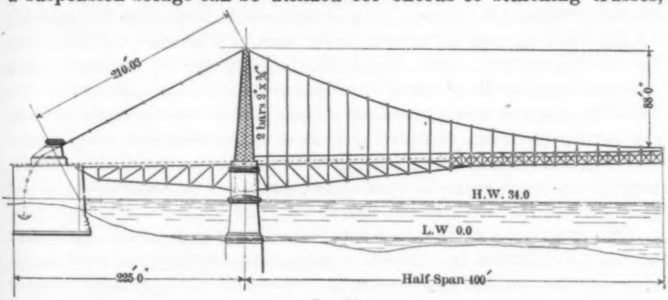

FIG. 26.

such an arrangement must naturally appeal to engineering instinct as the proper thing to provide, to say nothing of increased and substantial beauty and balance of the structure as a whole.

GUSTAVE KAUFMAN, M. Am. Soc. C. E.—The use of a cantilever in connection with the stiffening truss of a suspension bridge, as shown in Fig. 26, was contemplated by the writer several years ago. This plan was submitted by him to the authorities of the City of Pittsburg as a method for strengthening the Point Bridge at the mouth of the Monongahela River. In the event of its adoption by the city, some question might arise in the future as to its originality; therefore, to obviate any discussion on this point, its main features are shown here. The cantilever in this design is intended to relieve the cable of a large portion of its load; and it has been the writer's intention, since the conception of the design, to anchor the stiffening truss securely to the end of the cantilever, thus practically doing the same thing that has been proposed by the author. Mr. Kaufman.

Mr. Meem. J. C. MEEM, Assoc. M. Am. Soc. C. E.—One of the new and interesting features suggested by the author, that of substituting cables and backstays, socketed at the towers, for continuous cables such as have been used heretofore in practice, possesses undoubted advantages, and the question then arises as to whether or not there are disadvantages sufficient to offset these advantages. In this method of construction the fundamental consideration is the strength of the socket; for, if a socket can be constructed which shall be, under all conditions, stronger than the rope itself, then necessity for further discussion is eliminated. The author suggests, however, that the strength be calculated for the strength of the socket rather than that of the cable, and this would necessitate, in a weaker socket, such as noticed in the tests given in the paper, a greater weight of cable, assuming that the strength of cable and socket vary in the same relation. The sockets used in the tests were of the ordinary wedge type and broke always just inside, beginning with the outer wires; and this, of course, showed unequal pull in the wires, causing the wedges to pinch or shear them off. The author suggested that a more satisfactory socket could be, and in fact had been, made. One such was described by Mr. Spilsbury, which depended in part for its strength upon the fact that the outer wires were consecutively bent and run in behind the butts of the succeeding wedges which bore upon them. As the author bases one of his objections to the continuous cables on the fact that the wires bend in passing over the saddle, and as this objection to the bent wires is also raised against the ordinary form of socket, it would also apply to the new form.

The conclusion is, then, that it is necessary to depend on a socket of this kind, which has its objections, or else upon one of the wedge type which is weaker than the rope. When it is considered further that a socket is dependent upon stress to develop its full strength, and that any variation, however infinitesimal, in this stress, tends in this infinitesimal degree to impair its strength, and that a socket under the same conditions would doubtless be more liable to injury from atmospheric changes, and when to these two weakening factors of vibration and wear are added the others of bending and shearing in the socket, the conclusion is that this is the weak point in the cable. If, by substituting continuous cables, the proportional number of sockets can be reduced from six to two, it would seem best to do this, even if at a somewhat greater initial expense of time and labor.

Strength might be added to the continuous cable in its passage over the saddle, by enveloping it in a casing of iron or steel, between the interior of which and the cable were ball bearings or rollers clasping it firmly, yet admitting of longitudinal motion. This would, perhaps, develop most fully the strength of each wire or rope by preventing any tendency whatsoever towards their displacement.

T. Kennard Thomson, M. Am. Soc. C. E.—The author advocates using nickel steel, a metal supposed to undergo a permanent contraction at a temperature a few degrees below zero. As this change is sudden and must necessarily affect some parts of the bridge much sooner than others, it might be dangerous to use this metal in a bridge near New York. In a structure of this kind, where there is very little shock, it is a question whether it would not be safe to strain ordinary structural steel up to 20 000 lbs. per square inch. The writer was once ordered to design some movable short-span structures subject to very severe shocks, using a fiber strain of 20 000 lbs. per square inch. He refused absolutely to do this, and was allowed to go ahead in his own manner. He found afterward, however, that his structures were loaded to double the capacity used as a basis for the design, and many of them are still in service. Mr. Thomson.

George S. Morison, Past-President Am. Soc. C. E.—Before entering upon more specific subjects, the author would call attention to the fact that the design which is made the basis of this paper is, like the paper, a study rather than an adopted plan. He considered that in a work of this magnitude and novelty it was necessary, not only to prepare a general design, but to work out all the important details to an extent which would show that there were no insuperable obstacles in the way of their construction, and no defects which might materially increase the quantities or cost of the structure. He did not consider that such design was a final design, either in its proportions or dimensions; while he is confident that it would produce a satisfactory structure, the cost of which would not exceed the estimates, he does not claim that it is perfect, but believes that many changes would be made if the construction of such a work were actually intrusted to him. Mr. Morison.

The claim that a suspension bridge with cables in parallel planes is as well fitted to resist wind pressure as one in which the cables are cradled is strangely at variance with the principles which have generally governed suspension bridge construction. While it may be true that under extreme conditions the resistance offered to wind by a bridge with parallel cables will be about as great as that offered by a bridge with cradled cables, it must be remembered that the disturbances which give a bridge a bad name, which shorten its life, and generally are most objectionable, are not the maximum distortions which occur only at long intervals, but the minute and frequent oscillations which, if they occur at all, occur constantly. It was these oscillations which proved the greatest defect in the old iron bridges, and which accompanied, as they often were, by small rattling members, gave an appearance of insecurity which was too often well founded. Though the cradled cables may have no advantage to resist a hurricane which would blow the bridge 8 ft. out of line (a storm which would work

Mr. Morison. widespread havoc in the neighboring country and which has probably never extended over a few hundred feet at the same instant), they would have a very decided advantage in checking the oscillations, measured by fractions of an inch, which are caused by the passage of trains and due to other circumstances occurring every few minutes. Where the cables are in parallel vertical planes they, with the floor of the bridge, make a series of pendulums whose natural vibrations are perfectly synchronous and would readily become cumulative. On the other hand when the cables are cradled, each cable has a different motion, one goes up while the other goes down, and the motion of nearly every part of the structure greatly exceeds that of the center of gravity. The motions become complicated in so many different directions that their general effects neutralize each other, and the danger of cumulative synchronous effects, if not entirely eliminated, is very greatly reduced. A very simple experiment illustrates this difference. Suspend a horizontal bar by two parallel cords from a fixed bar above, set it vibrating in the direction of its length and observe the time which it takes for the vibrations to cease; then spread the cords apart at the top, so that their positions correspond to the planes of cradled cables, set it vibrating again, and observe the time it takes for vibrations of equal amplitude to cease. The time decreases rapidly with an increase of cradling.

If a stiffening truss of excessive depth is to be used, the central hinge is a matter of necessity; in no other way can the deflections of the cables due to strain and temperature be taken up without unreasonable stresses in the chords of the stiffening truss. Stiffening trusses with central hinges are described, and mathematical demonstrations of the strains in them given, in nearly all the text books which have treated of suspension bridges. They have seldom been built. There are two very serious objections to the central hinge. It is at best a loose detail in the point where stiffness is most needed, giving an opportunity for lost motion at the very point where the tendency to oscillation is greatest. The amount of motion on the central hinge is considerable, amounting in this case to something like $1^\circ 20'$, and if the stiffening truss is to be sustained by the cables throughout its length, the same bending motion must be concentrated at or near the central point of each cable; such concentration of bending motions would prevent the use of cables of large dimensions and perhaps would require a hinge in the cables as well as in the stiffening truss. A hinge which will permit of the necessary amount of motion and be adequate to carry the strains, both tensile and compressive, which will exist in a structure of this magnitude is believed to present mechanical difficulties which are practically insuperable. To obviate this difficulty it is proposed to make a hinge capable of resisting neither tension nor compression, using ropes or flexible plates in approximately vertical posi-

tions. A hinge, if that is a proper name to apply to such a device, can be made in this way which will transfer vertical reactions, but it will be capable of no other duties. The whole value of the chords of the stiffening truss as members of a wind truss is thrown away, the chords being cut at the point where the wind strains are greatest. The wind must be resisted entirely by the cables; and cables, placed in vertical planes, can resist the wind only by swinging out of line until the suspended superstructure has been lifted to a point where its weight balances the wind. This will require a lateral deflection of 20 ft., instead of 8 ft., which seems fatal to this design. Mr. Morison.

The application of mathematical demonstration to test the relative merits of complicated designs before the details of those designs have been worked out is perhaps the most delusive abuse to which mathematical demonstration can be applied. It is believed that when the actual details of construction of the plan proposed by Mr. Mayer are developed with the same care that has been done in this paper, the plan will be modified in some very essential features.

The versed sine of one-eighth the span was selected largely because this ratio was used in the reports made by the two Boards of Engineers which have considered the subject of suspension bridges of this magnitude. It was not selected because the author believed it to be the most economical ratio. It was the natural ratio to select for a first study; but, before adopting any final design for construction, other ratios should be studied with equal care. The author believes that a less versed sine would be both more economical and in other ways better, but this would not affect the general details of construction, nor modify the cost enough to affect the general feasibility of such a bridge.

In like manner a deflection of 3.5 ft. was adopted as the permissible deflection in a length of 1 400 ft., because it corresponded to a 1% grade at each end of the curve. In view of the fact that the maximum deflection would occur only under a condition of loading which, though possible, is very improbable, and as trains of sufficient length to produce this deflection would balance themselves, one portion descending while another portion is ascending, the author believes that a deflection of twice this amount, or 7 ft., would be entirely unobjectionable. He would be prepared to recommend a permissible deflection of 5.25 ft., corresponding to a grade of 1.5% at each end of the vertical curve.

A reduction of the versed sine from one-eighth to one-tenth would, roughly, increase the weight of the cables 25 per cent. It would increase the portion of excess load distributed by the cables from 9.424% of the dead load to 11.78%; while it may be roughly stated that an increase of the permitted deflection from 3.5 ft. to 5.25 ft. would further increase the amount of excess load one-half, so that the portion of the

Mr. Morison excess load absorbed by the cables would be 17.67% of the dead load; as the dead load is 39 000 lbs., this amounts to 6 891 lbs., which, deducted from 12 000 lbs., leaves 5 109 lbs. to be distributed by the stiffening truss, as against 8 325 in the design described in the paper. The depth of the stiffening truss, which is determined by the permitted deflection, would be reduced from 66 to 44 ft., and the theoretical section of the chords from 450 to 418 sq. ins. This means, in a general way, an increase of 25% in the weight of the cables, a decrease of 40% in the webs of the stiffening trusses, and a decrease of 7% in the chords of the stiffening trusses.

If the versed sine were reduced to one-twelfth and the permissible deflection increased to 7 ft., the effect would be, roughly, to increase the percentage of excess load distributed by cable to 28%, or 10 920 lbs. per lineal foot, leaving only 1 080 lbs. to be distributed by the stiffening truss, while the depth of the stiffening truss would be reduced to 33 ft. Such a design would probably be adopted for a highway bridge of these dimensions.

These illustrations are given as showing the modifications which are admissible in a design like that described in the paper, and which would be carefully considered before any such design is actually constructed.

It has been said that if distortions do not occur, the force which would produce the distortions must be absorbed somewhere in the structure. With trains running at high speeds, it is probable that the deflections would be reduced. The inertia of matter, which has comparatively little to do in small structures, would be important in a suspension bridge like the one described. The force of gravity is constant, but time is essential to the operation even of gravity. If gravity can be applied to only one-tenth the mass it has to move (as in the familiar experiment in which a weight of 4½ lbs. is placed at one end of rope passing over a pulley and 5½ lbs. at the other) it will take ten times as long for gravity to move this mass a given distance as it would to move the one-tenth to which it is actually applied. The same rule will apply to a bridge of this kind, with a limited time in which to create deflections. The deflections from light loads passing at high speeds will be less than from the same loads in a static position ; this means that trains which run at high speeds may have comparatively little effects on the great structure.

One of the principal features of the design is the use of wire ropes instead of independent wires to make the cables. These ropes would perhaps more properly be called strands. They are not twisted ropes of the ordinary character, but resemble the strands of which an ordinary rope is made up. Such strands are now made for various commercial purposes. They were selected some years ago by General Serrell as the best material to use for this purpose in his proposed

bridge across the Hudson River at Fort Montgomery. The author believes that strands could be made much better adapted to this purpose than any which have yet been manufactured. In his first studies he contemplated making these strands of straight wire, but abandoned the idea because he did not believe that it was possible to transport them. A straight wire strand can only be handled when under a very considerable strain, and in strands of this length this strain can be maintained only by using the weight of the strand to produce it. Before actually manufacturing strands for use, experiments should be made to determine how small a twist is needed to insure the stability of the strand during transportation, and the author fully believes that this twist can be reduced to such a degree that the difference in strength between the twisted strands and the straight wire will be exceedingly small. Mr. Morison.

The experience of the East River Bridge is cited to show the necessity of adjustment of the different strands at the time they are placed. The author admits that such adjustment may be required under the sockets, but he believes that sufficiently accurate work can be done to render it unnecessary. He remembers the time when it was considered necessary to put adjustments in all the tension members of the webs of otherwise pin-connected trusses. A certain amount of error always exists and is permissible in every structure; the main cable is about 40 000 ins. long, and an error of 1 in. in the length of this cable would correspond to a variation of less than 700 lbs. per square inch in the strain on that cable; a variation of several times this amount would be taken up by the friction of the ropes upon each other. The author, however, would never think of using ropes in this way until those ropes had been actually subjected to a strain at least 10 000 lbs. greater per square inch than anything they would be expected to bear in the bridge, and had been kept under this strain for at least two days. This would be necessary to make sure of the uniform elongations which the tests made at Watertown show can be obtained.

To accomplish this he had contemplated building two substantial masonry piers at proper distances apart at the works where the ropes are manufactured, the space between these piers to be covered with a light shed, to keep out both rain and sun. On these masonry piers saddles should be securely fixed with bearings for the sockets, while suitable machinery should be provided for stretching the ropes under strain between these piers. The actual strains in the strands would be determined by measuring the deflection. Every rope, before it is shipped to the bridge, would be placed on these piers, adjusted under strain to the proper length, the sockets fitted on and trimmed by a special tool which would turn the bearing after the socket is on the rope, so that the deflection would correspond to the exact strain de-

Mr. Morison. sired in the rope. These piers should be made of such dimensions that twenty or more ropes could be placed there at the same time, and the deflections of the ropes first placed would be used to measure the deflections of the later ones. No rope should remain in this testing shed less than two days. After testing, each rope should be rolled up in a coil of considerable diameter (perhaps too large to transport on a railroad car) so that the compacting effect of the strain would not be modified by the coiling of the rope. The author fully recognizes the fact that the elongation which would occur in a twisted rope which had not been subjected to strain in this manner might be fatal to anything like uniformity in the action of the cables. By this method he believes that these difficulties can be removed.

The ropes used as suspenders, though of the same weight and sections as those used in the main cable, are subjected to less than half the same strain per square inch; and as they pass around saddles of about 40 ins. diameter, it would probably be expedient to use softer wire in them than in the main cables.

The sockets at the ends of the cables may be elements of weakness, but they do not necessarily mean more than a screw at the end of a rod or the heads at each end of an eye-bar. The same kind of objections that are now raised to a socket on a wire rope were raised thirty years ago to an upset head on an eye-bar, when some of the best engineers absolutely refused to allow eye-bars to be forged by a process which so weakened the fiber of the iron, and welds in the body of the bars were preferred by many to any upset enlargement for a screw. It is only within the last ten years that there has been anything like a reasonable certainty of securing eye-bars in which the head would break the body of the bar, but engineers accepted upset bars as better and safer than any welded bars that could be made, recognizing that while the heads were not what they would have been glad to have, the fractures, which they could not avoid, occurred under strains far above any working strains and in no way affected the elastic limit.

The change which is now going on in the manufacture of sockets is of like kind. Mr. Spilsbury states positively that he can make a socket which will develop the full strength of the rope; even if this were not so, the strength of the rope in the socket can undoubtedly be increased to a limit which will compare favorably with the usual strength of eye-bar heads, and which will be so far in excess of any working strains that the weakness is more a matter of theory than of practice. From its very nature the weakness of a socket is developed principally when the contraction of the wires under extreme strain prevents their filling the full area of the socket, and in hard steel wires this contraction occurs only at strains which are relatively far in excess of the elastic limit in steel and iron bars.

Wire ropes are used in many places where the strength of a socket is absolutely essential to safety. A single rope used for a carrier in a wire tram is dependent on two sockets, the failure of either of which would produce disaster. In the cables of the proposed bridge there are 253 ropes, and the risk is divided among a corresponding number of sockets, the failure of several of which might occur with no serious result. Mr. Morison.

There is no question but what the detail adopted for the connections at the top of the tower is heavy and expensive. Practically the entire weight of these special connections would be saved if the cables were made continuous from anchorage to anchorage. Furthermore, if the side spans were suspended from the backstay cables, those cables could be made to terminate horizontally in the anchorage, thus reducing the strains on the socketed connections more than 10%, when the versed sine is one-eighth, below the maximum strains in the cables. There were, however, several reasons which made the author prefer to use the detail described in the paper.

A tower which is unsupported laterally at the top requires a large base to secure sufficient stability; on the other hand, a tower which is held at the top becomes simply a post, like the guyed mast of a derrick, which may be made of much smaller dimensions. The only way to guy such a slender tower satisfactorily is to fasten the cables rigidly to the top. The detail adopted holds the top of the tower rigidly and further makes equal inclinations of main cables and backstays unnecessary. Where there is no special reason for doing otherwise, equal inclinations would probably be selected, but where land is valuable the cost of a site for an anchorage might be greatly reduced by increasing or decreasing the inclination of the backstays. The friction of cables passing over a curved saddle would probably be enough to hold the top of the tower securely, but engineers hesitate to depend on friction in places where a slight movement against friction would produce an injury which could not be repaired. In spans of moderate length where the two inclinations are equal, continuous cables might be the best practice. In such spans as the one considered, the author would hesitate to use them unless the towers were made of such dimensions as to depend entirely on the base for their stability, and this involves movable saddles.

The plan designed simply suspends independent ropes between two points. The strains in these ropes can be determined by their deflections, thus insuring practically uniform strains in the several ropes which compose the cable. If the ropes were to be run continuously from anchorage to anchorage, they would be suspended at four points, and their free motion would be affected by the friction on the bearings over the towers. It would be possible to adjust them to very nearly uniform strains, but the time required for such

Mr. Morison. adjustment would be considerable, and the danger that an occasional rope, supported for its whole length on the ropes below, would be slack without the slackness being observed, would be very greatly increased.

It is absolutely necessary that every rope should be strained, as already explained, before it is put in the bridge. The difficulty of subjecting ropes to such strains increases rapidly with the length of the ropes. A deflection of 66.5 ft. corresponds to a strain of 70 000 lbs. per square inch of a rope 3 312 ft. long, the length of the main cable; whereas, the deflection corresponding to the same strain in a rope 5 878 ft. long, the total length from anchorage to anchorage, is 205.7 ft. The towers and sheds required for the former test would be comparatively simple structures of practical dimensions. The towers and sheds required for the latter test would be of prohibitory size. It would be necessary to resort to the complications of intermediate supports with the uncertainties which would attend their use.

The connection between the two sets of cables at the top of the tower would have to be very carefully made, and involves some difficulties of construction. The author at first contemplated using single pins in place of the 5-in. bolts, and discarded them simply because of the large dimensions which these pins made necessary. The vertical members which hold down the lower ties could be slackened after the upper ties are in position, so that the strains would be divided between the two sets of ties as designed, by the principles of leverage, in an inverse ratio to the distances of the ties above and below the line connecting the theoretical intersection points. The duty of the several 5-in. bolts corresponds very largely to the duties of rivets in riveted joints. Accurate work would be required, but accurate work can now be had, and with the present facilities for electric heating of local members, connections can be made which a few years ago were impossible. The long pins or bolts could not be driven by the battering ram usually used in bridge erection, but must be forced in slowly by hydraulic or other extreme pressure. While the author admits that the construction of these connections is a nice piece of work, he has no doubt of the ability to carry it out in a manner which would produce satisfactory results.

The unequal length which the splaying out of the ropes to form the connections will make in the length of the separate ropes can easily be provided for by varying the depths of the holes in washer plates into which the sockets fit. The point raised by Mr. Lindenthal, that the strains in the outer ropes would be seriously increased by the cradling of the cables if these are first strung in vertical planes, is well taken. The author confesses that the calculation of the strains which would be produced in this manner has given results which he

had not expected. The plans described in the paper should be modified to the extent of laying the cables originally in their cradled position, at least through the splayed portion, while it would probably be expedient to continue the cradling a considerable distance further. This could be accomplished by connecting together the iron horses in which the opposite cables are laid. It would also be expedient to stop the wrapping 100 or 150 ft. from the top of the tower instead of 50 ft., as shown in the design, enclosing the intermediate portion of the cable in a steel envelope somewhat similar to that which Mr. Lindenthal has proposed to use for his North River Bridge. By this change the increase of strain in the upper ropes caused by the deflection of the cables would be very much reduced. The extreme estimated deflection below the normal position at the center of the span is calculated as 7.11 ft., which corresponds to 0.46 ft. at the end of a tangent 50 ft. long, and to 1.38 ft. at the end of a tangent 150 ft. long. The former corresponds approximately to a variation of 10 500 lbs. per square inch in strain on the outside ropes, and the latter to a variation of 4 200 lbs., which, occurring only in very warm weather, is well within permissible limits. Mr. Morison.

The author recognizes that the deflections of the towers during construction would be considerable, and that they would be much greater if no weight is carried by the backstays than if the backstays support the side spans. It would, of course, be possible to take up this deflection by bending the tower, additional material being provided to resist such bending; but the author does not believe that a careful estimate would show this method to be economical, and it would probably involve using a more slender tower than has been designed, the base having been fixed in a measure by the necessary dimensions of the foundations, including proper space to work between the several independent cylinders. In a small bridge, in which the number of foundations could be reduced, this method would readily be adopted, or the tower could be hinged at the bottom. The author has had in mind a somewhat original method of getting over this difficulty, and, that is, to support the posts of the tower on the river side on cylinders filled with lead, which lead would be under a pressure far in excess of that under which lead flows. By tapping holes in these cylinders the lead would be allowed to flow out and the posts would settle to their true position, the flow of the lead being so slow that the settlement could easily be checked or regulated. In other words, the posts would rest temporarily on hydraulic presses, in which the liquid used is lead.

As regards the connection at the bottom of the anchorage, where no rock exists in which to place the auxiliary anchorage for use during construction, it would either be necessary to provide additional masonry or make some other temporary device by which the plates would be

Mr. Morison. held in position. The author does not regard the necessary transmission of a considerable pressure through green masonry as a serious thing. It would once have been so, but a mortar made with good Portland cement becomes hard in so short a time that no trouble need be apprehended from this source. On one occasion he constructed a number of small brick piers, with steel rods passing through those piers from base to top, and instructed the resident engineer in charge to screw up these rods until there was a strain on them of 15 000 lbs. to the inch. The resident engineer, expecting compression in the piers, built them slightly higher than the plans called for, and on their completion immediately screwed up the rods until the elongation shown by the screw threads indicated that the necessary strain was developed. The piers continue to this day as much higher than the plan as he had built them.

No final specification has been drawn for the steel of a bridge like the one described in the paper. The author thought it best to provide for nickel steel for the portions of the chords which are subjected to the greatest reversal of strains; one of his reasons for doing so was the objections which he knew would be raised to the use of ordinary steel under these conditions. So far as the author knows, experiments made on high carbon steel, under conditions in which strains are frequently reversed, show an ability to resist such reverse strains about equal to the ability of nickel steel having the same elastic limit. He believes that this bridge would be perfectly safe for the service which it has been supposed to perform if high carbon steel without nickel were used in the chords. He would prefer, however, not to take the responsibility of using what is generally considered a brittle material when a tougher material of equal strength is available, though at greater cost. The case of the St. Louis Bridge is cited to show that a very hard steel which is sometimes subjected to tension as well as compression, has resisted unusually high strains for 22 years without showing any defect; if it could be proved that no fractures exist in this steel the author would consider this strong evidence in favor of the use of high steel under correspondingly high strains, but unfortunately no one knows the present condition of the steel in the arches of the St. Louis Bridge. Each section of arch is formed of six staves of hard, high carbon steel (not chrome steel as is commonly reported), enclosed in a riveted jacket of softer material; the staves are utterly inaccessible, and fractures may have occurred without being known.

The author recognizes the objection which may be made to a cross-bracing between the two stiffening trusses which is not sufficiently strong. The bracing used depends on the stiffness of the broad web members between the connection points and the bottom chords, a practice which is in general use in all through bridges. He would prefer to extend his diagonals to the bottom chord, but the arrange-

ment shown will give almost equally good results, provided the web members have proper lateral stiffness. Mr. Morison.

The study gives the design of an engineering structure, a tool intended to do work, and not a monumental design for ornament or commemoration; undoubtedly some of the features might be modified in a way which would enhance beauty. The author thinks that such modification would have been out of place in a paper like the one under discussion, "submitted with a view to opening the way for improvement and to show that a great suspension bridge, which would be well adapted to railroad service, would involve no insurmountable difficulties of construction." He has, however, long felt that no object can be really beautiful in which adaptation to its purpose and correct construction are not the fundamental features of the design. Ornamentation may mar or beautify according to the skill with which it is done, but the artist who knows nothing of construction is no more competent to pass on the beauty of a great bridge than is the man without musical education to criticise the classical works of great composers. No decoration however bad, unless it conceals all constructive outlines, can destroy the beauty of a correctly designed and proportioned work, and no decoration however good can render beautiful a work built on poor lines of construction. Nature may abhor straight lines, but Nature has built with an excess of material which engineers cannot have, and yet in the needles of a pine forest, in the trunk of the palm tree and in the prisms of columnar basalt, it has shown that it has use for straight lines. A curved line is unfitted to resist either tension or compression unless accompanied by modifying transverse strains; a tower held at both top and bottom is simply a compression member which construction demands shall be straight and whose real beauty would be destroyed if curved lines were introduced. The design advocated by Mr. Mayer substitutes curves for the straight lines of the top chord of the stiffening truss selected by the author; if a comparison be made of the beauty of the two structures, the author has no fear of the result.

Two monuments, both bearing the name of Washington, have been built which illustrate the author's idea of beauty so well that they may be cited here. One stands in the city which also bears his name, a single upright shaft, perfectly plain, every line of which is straight; it was built by an engineer; it stands there with no other beauty than that given to it by its perfect proportions, one of the few of man's works which looks as if it might have had a divine origin, like a single index finger pointed to heaven. The other is in New York, in a square which also bears the name of Washington. It is an arch in which all the principles of monumental construction are violated by making the abutments so thin that the first thought of the observer is of the effort they must make to resist the thrust of the arch. Where perpetual rest

Mr. Morison. should prevail, we find the torture of eternal strain; exquisite decoration lavishly applied fails to cure the false proportions, and the curved arch of the architect is as ugly as the straight obelisk of the engineer is beautiful.